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CONSTRUCTIONAL ENGINEERING

UNIV. OF MICHIGAN

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EDITORIAL NOTES

Architects and Contractors.

CONCERN has been expressed by members of the Royal Institute of British Architects in private practice because some contractors now employ architects on their staffs and undertake both the design and erection of structures. This is known as an " all-in " service, and is similar to the practice in the U.S.A. and elsewhere where some firms employ architects, structural engineers, and other specialists and undertake the design and erection of structures. It is also akin to the practice in some Continental countries where an engineer (often with a world-wide reputation) acts as architect and engineer and is also a director of the firm that undertakes the construction for a sum that includes the cost of the architectural and the structural design. The by-passing of consulting engineers in private practice has for many years been acceptable to the architectural profession, who see no objection to the cost of professional engineering services being included in the estimates of contractors or of sub-contractors, in the same way that the contractor who offers an " all-in " service includes the cost of professional architectural services by his employees in his estimate for a structure.

A committee appointed by the Royal Institute heard evidence from some of the contractors who offer an " all-in " service and from architects employed by them. The report of this committee expresses the view that the Royal Institute should not attempt to prevent the employment of architects by contractors, although the Northern Architectural Association emphatically recommended that the " all-in " service be abandoned. Architects in private practice would find it difficult to criticise the architectural merit (or lack of merit) of the buildings produced by contractors who offer an " all-in " service, because architects employed by contractors must be registered or they could not be described as architects, and many of them have passed the examinations of the Royal Institute. The assumption that all men who have passed the same examinations are equally competent is a fiction that apparently has to be maintained. Nothing is likely to be farther from the truth ; an architect or an engineer employed by a contractor, a Government department, a local authority, or in industry may be as good as, or better than, or worse than an architect or engineer in private practice who has passed the same examinations.

The chief reason given for disliking the " all-in " service is that it upsets the

traditional claim that the architect is the leader of the building team and is an independent arbiter, with some legal authority, between the builder and the building owner. This, of course, is true. An architect employed by a contractor can prepare designs and recommend methods of construction and materials, but in this case the final decision lies with the contractor who is concerned to submit a competitive tender in order to secure a contract. But is the result of this procedure likely to be very different from that used in the design by private architects of many buildings to-day? The advantages of early collaboration between all concerned with the design and erection of a building have been much emphasised in recent years, and such collaboration often takes place. In the case of a contractor's " all-in " service those responsible for the design and construction and the estimate of cost work together in the same office, and such collaboration is a matter of course—indeed it is claimed to be the chief reason for the cheaper buildings that are said to result. In the case of a private architect, suggestions made by engineers that result in more economical planning or the better use of materials or the use of different materials, and suggestions made by the contractor to permit the best use of his plant, could result in so many alterations to the architect's original conception that it might be hardly recognisable in the completed building. If the private architect and the contractor's staff architect are equally good architects the result is likely to be much the same.

The " all-in " service in which the architect is the servant of the contractor changes the traditional relationship between an architect and a building owner. Such an architect is not independent and is no longer the representative of the building owner, who is deprived of what has been considered to be an important safeguard in the form of an agent recognised by law who acts on his behalf, even to the extent of withholding payment if he does not consider that the work has been done properly and in accordance with the specification. This is an important factor, but it need not be unduly emphasised. In such " all-in " contracts it is possible for the building owner to appoint a private architect or other agent to look after his interests.

There is no doubt that the committee acted sensibly in recommending that no action be taken in this matter. Whether a building be designed by a private architect or by one employed by a contractor it generally has to be approved by a local authority ; this is some safeguard against bad design, although some of the buildings designed by private and official architects indicate that it is not always sufficient. Provided that the contractor employs competent architects the safeguard of the building owner lies in the integrity of the contractor, as it does when an industrial structure is designed and built by the same firm and no architect or consulting engineer is employed. And when the building owner employs an independent architect or surveyor it is surely no disadvantage that his representative is able to consult with an architect employed by the contractor who knows the problems of the architect as well as those of the contractor.

Some of the most eminent consultants in reinforced concrete were formerly salaried engineers of the staffs of firms who act as designers and contractors. It may be that some of the best of the architects now employed by contractors will become private practitioners, and have the advantage of the knowledge of modern materials and building methods and of business which are said to be now lacking in so many private architects and to be one of the principal reasons for delays in construction. This could be of much value to the architectural profession.

Precast Concrete in a School Building.

SLENDER PRESTRESSED COLUMNS 43 FT. HIGH.

A NEW secondary school, built for the London County Council at Peckham, comprises six main structures (*Fig.* 1) providing places for 1590 pupils and a teaching staff of about eighty. The plan was influenced by the need for economy and the desire to preserve some of the trees on the site.

The school includes an assembly hall, classrooms, and an administration building which includes a library, an art department and a gymnasium. The floor area is 68 sq. ft. per pupil ; the statutory requirement is 65 to 75 sq. ft. The buildings are arranged around the assembly hall from which there is direct access to the two classroom buildings, the administration building, and the gymnasium. There is a separate two-story building for housecraft classes.

A feature of the school is the use of prestressed precast concrete in the classroom buildings, the administration building, and the assembly hall. The first three are all built to the same basic design (*Fig.* 2). One of the classroom buildings and the administration building are three stories high (*Fig.* 3) and the other classroom building has a partial

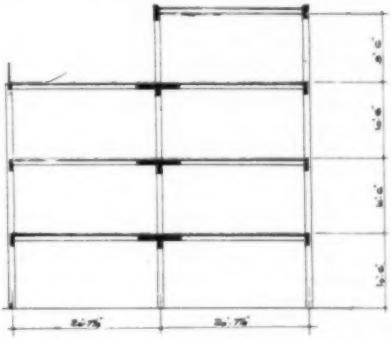


Fig. 2.—Typical Section through a Classroom Building.

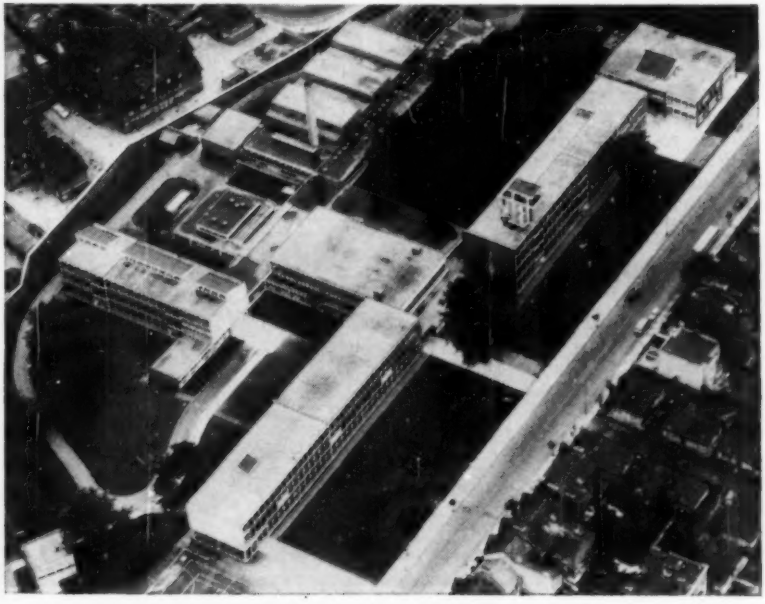


Fig. 1.—Aerial View.

fourth story with a tank room above (*Fig.* 1).

All these buildings have a central line of columns and a spine-beam of reinforced concrete cast in place, and prestressed precast columns and edge-beams in the external walls. At the intermediate floor levels the precast edge-beams are covered on the outside (*Fig.* 4). Prestressed beams 5 in. wide at 21 in. centres span 20 ft. 6 in. between the edge-beams and the spine-beam at each floor level. The gaps between these beams are bridged by foamed-slag concrete slabs 2 in. thick which form the permanent shuttering to a reinforced concrete topping 2 in. thick. Reinforcement placed over the top of the spine-beam provides continuity between the two bays.

The external precast columns are 9 in. by 6 in. in cross section and spaced at 6 ft. 2¼ in. centres. The internal columns are 9 in. by 9 in in cross section and are generally at 12 ft. 4½ in. centres, but at some places alternate columns are omitted, in which case the span of the spine-beam is 24 ft. 9 in.

Gable beams and end columns were cast in place, as also were the staircases which are in bays in the rear half of the buildings, 25 ft. from the end walls, and in a bay near the centre; in the administration building there are only two staircases. The overall lengths of the three buildings are 186 ft., 211 ft., and 125 ft. respectively.

The arts rooms in the administration building have north-light roofs over half of their area except at the stair-wells.

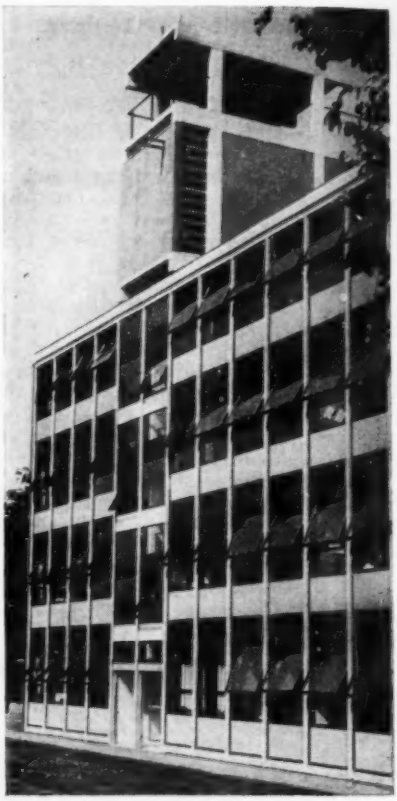


Fig. 4.—Four-story Part of a Classroom Building.

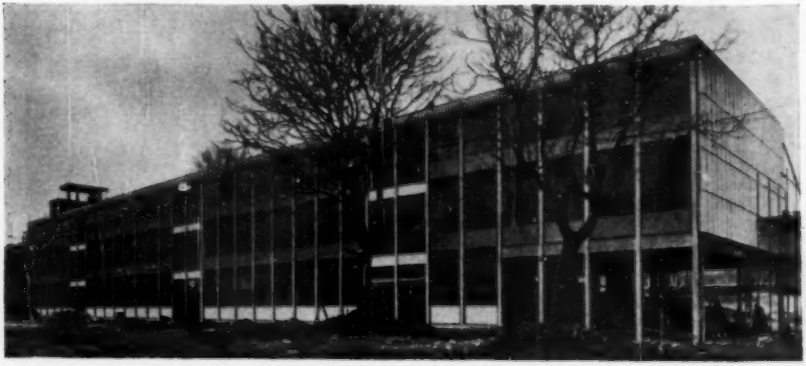


Fig. 3.—Rear of a Classroom Building.

The north-lights are formed by precast cranked beams which support steel-framed glazing and a timber and felted roof. With the exception of the lower roof in one of the classroom buildings (*Fig.* 2), prefabricated timber was used for all the flat roofs.

The main consideration in the design of the external precast columns was that they should have a uniform surface without joints (*Figs.* 3 and 4). This necessitated the casting of columns 33 ft. and 43 ft. in length. Because stresses occurring during handling would cause cracking in such columns if they were of reinforced concrete they were prestressed. The cheapest method would have been the pre-tensioning process with wires, but

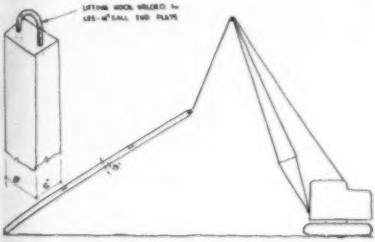


Fig. 5.—Method of Lifting Columns.

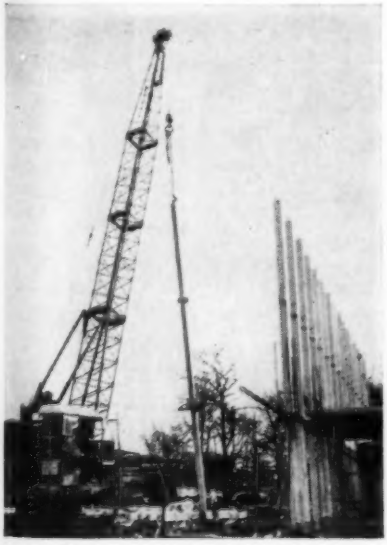


Fig. 6.—Erecting Columns.

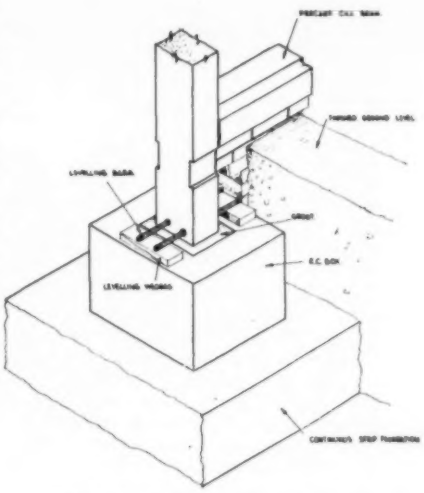


Fig. 7.—Foundation of Column.

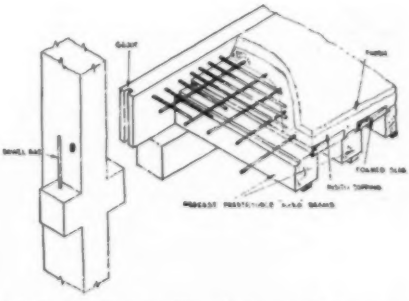


Fig. 8.—Connection between Column, Beam, and Floor.

it was considered to be doubtful whether absolutely straight units with a slenderness ratio of 86 could be made in this way since slight eccentricity of the prestressing wires would have caused winding or twisting.

Each column is therefore prestressed with one post-tensioned Lee-McCall bar at its centre. Bars of $\frac{3}{4}$ in. diameter were used for the 3-story columns and of 1 in. diameter for the 4-story columns. The bars were well coated with bitumen and secured in position before the concrete was cast. The bitumen prevented the development of bond, and it was possible to apply the specified tensioning force without causing lateral deformation.

The columns were provided with lifting hooks at the top and lifted by a crane (*Figs.* 5 and 6) which lowered them into prepared bases formed in a continuous strip foundation, and they were aligned by means of levelling bars and wedges (*Fig.* 7). The prestressed edge-beams rest on brackets on the columns and are secured in position by a dowel-bar which fitted into a groove in the beam and which was grouted in (*Fig.* 8), allowance being made for slight inaccuracies in casting and setting out (*Fig.* 9). The next stage was the erection of the floor beams (*Figs.* 10 and 11), which are connected to the edge-beams by protruding links embedded in the topping (*Fig.* 12).

In each of the three buildings one expansion joint is provided with double columns, which were temporarily clamped together (*Fig.* 13). The columns are anchored in the topping at each floor level by means of a looped bar threaded through a shaped ferrule cast into the columns (*Fig.* 12). A different method of anchorage was used at the columns adjacent to the stair-wells, where the columns were tied into the half-landings by a bar screwed into a cast-in ferrule (*Fig.* 14).

At roof level the precast edge-beams and coping units are connected to one another and to the columns by reinforcement bars, all grouted in, as shown in *Fig.* 15. The precast copings have a mineralite facing on the outside and a

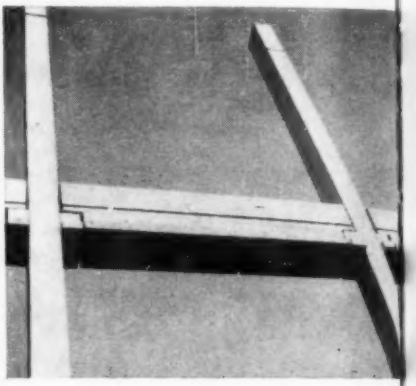


Fig. 9.—Joints of Beams and Columns.

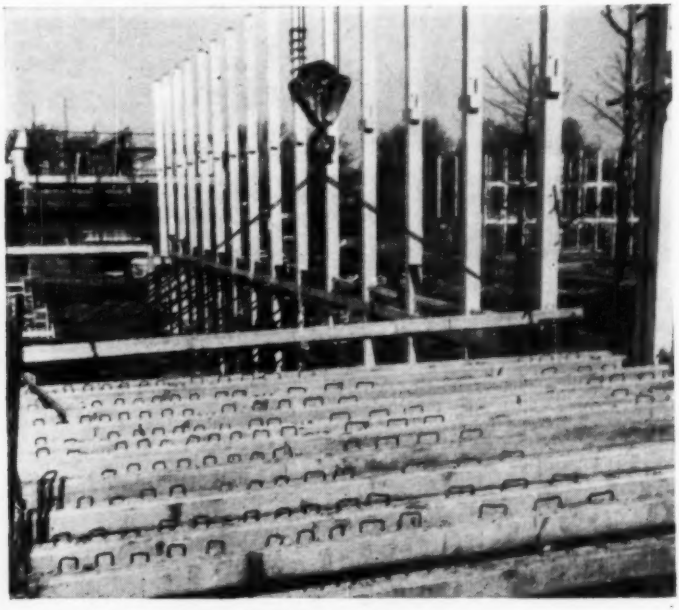


Fig. 10.—Erecting Floor Beams.

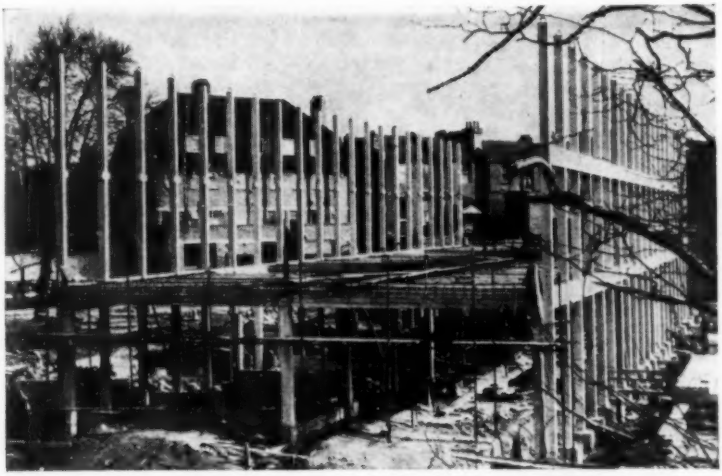


Fig. 11.—Erecting Columns, Edge-beams, and Floor Beams.

timber rail on the inside to support the prefabricated wooden roof. The edge-beams are tied across the building by reinforced concrete beams cast in alternate troughs between the timber floor units at 12 ft. 4½ in. centres (*Fig.* 16). The timber was lined with bituminous felt to avoid staining the soffit. A detail of the support of the cranked roof beams in the administration building, where a hole had to be left for the rainwater pipe, is shown in *Fig.* 17.

The assembly hall is 80 ft. long by 58 ft. wide and 20 ft. 3 in. from ground level to ceiling, with a floor 3 ft. below ground level for most of its length. The width is spanned by prestressed precast I-beams

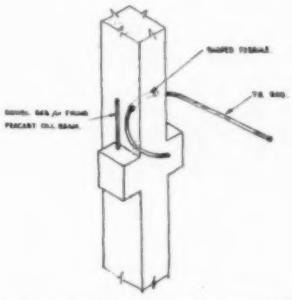


Fig. 12.—Connection between Precast Column and Floor Topping.

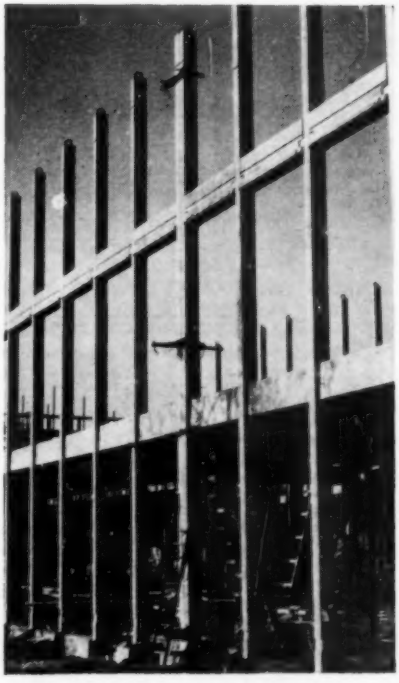


Fig. 13.—Double Columns at Expansion Joint.

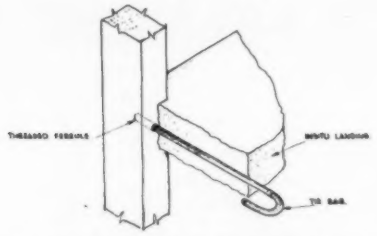


Fig. 14.—Connection between Precast Column and Landing Cast in Place.

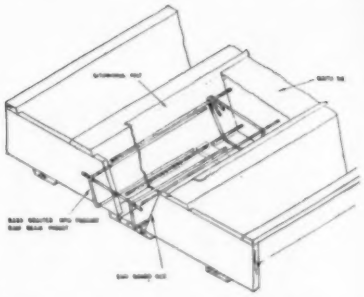


Fig. 16.—Connection of Edge-beam and Tie at Roof Level.

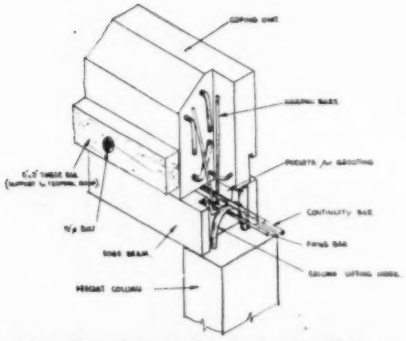


Fig. 15.—Connection between Edge-beam, Coping, and Column.

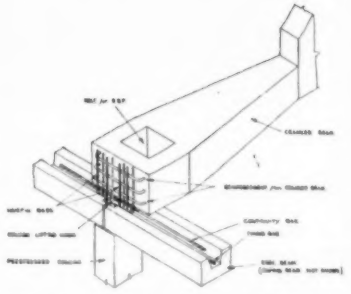


Fig. 17.—Connection at Roof Level between Column, Edge-beam, and Cranked Beam in Administration Building.

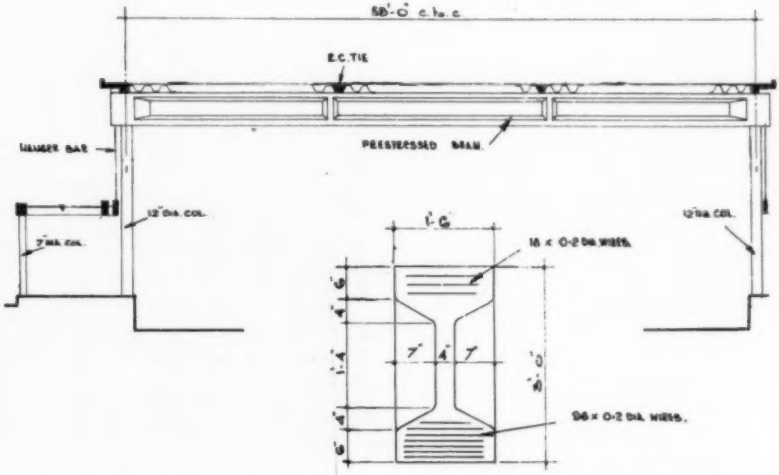


Fig. 18.—Cross Section of Assembly Hall and Prestressed Beam.

3 ft. deep and 1 ft. 6 in. wide supported on columns of 1 ft. diameter at 20 ft. centres.

The beams are prestressed in the flanges by 114 wires of 0·2 in. diameter at vertical and horizontal spacings of 1 in. (*Fig.* 18). The beams bear on steel plates, each with two threaded holes, cast on the tops of the columns, and are fixed by two bolts screwed through threaded holes in steel plates at the ends of the beams and the holes in the plates on top of the columns. The beams weigh 15 tons and were lifted by two cranes, one at each end (*Fig.* 19). On two sides of the hall there are corridors 9 ft. and 10 ft. wide by 7 ft. 6 in. high.

The reinforced concrete roof beams carrying a prefabricated wooden deck are suspended on one side by 1-in. diameter bars from the cantilevered ends of the prestressed roof beams of the assembly hall and supported on the other side on reinforced concrete columns cast in place. On the south side these columns also support the edge-beams which carry precast roof beams over the dining hall, which has a span of 30 ft. and is 100 ft. long.

Fig. 19.—Erecting Beam in Assembly Hall.

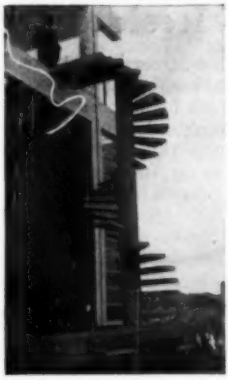


Fig. 20.—Precast Staircase.

Fig. 20 shows a spiral staircase on the roof of one of the classroom buildings; this is formed of precast reinforced members each comprising a ring and a step cast monolithically. These are arranged one above another and reinforcement bars and concrete were placed in the cylinder formed by the rings.

It is considered that the ease and speed of erection of all the precast units have fully justified the unusual design. The architects were Messrs. Lyons, Israel & Ellis, the consulting engineers Messrs. Hajnal & Myers, and the contractors Messrs. W. J. Simms, Sons & Cooke, Ltd. The precast ("Myko") floor beams were made by Messrs. George Greenwood & Sons, Ltd., and the Concrete Development Co., Ltd. The prestressed beams for the assembly hall were made by the Concrete Development Co., Ltd. The precast columns and other precast beams and the precast spiral staircase were made by the Liverpool Artificial Stone Co., Ltd.

Residential Flats in East London.

At Claredale Street in east London structures containing 164 maisonettes and eight flats are being built for the Bethnal Green Borough Council.

One of the buildings is fifteen stories high and is in the form of four wings radiating from a central structure containing a staircase and two lifts connected to the wings by bridges at alternate floor levels. The structure is entirely of reinforced concrete except that the floors of the living rooms of the maisonettes in alternate stories are of timber. The fronts of the balconies and the edge-beams were precast and delivered to the site with finishes already applied. The foundation is of raft construction in ballast about 9 ft. below ground level, and the boiler-house, fuel stores, and tenants' stores are accommodated within this height.

There are also two buildings of six stories (*Fig.* 2), with cross walls 6 in. thick supporting concrete floors at living-room levels and timber floors at bedroom levels. The precast elements were made on the site, and included stair flights, edge-beams and floor slabs 17 ft. 6 in. long by 4 ft. wide.

The architects are Messrs. Fry, Drew, Drake & Lasdun, the consulting engineers are Messrs. Ove Arup & Partners, and the contractors Messrs. Wates, Ltd.

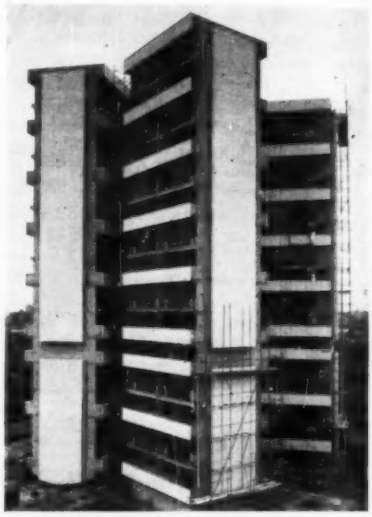


Fig. 1.

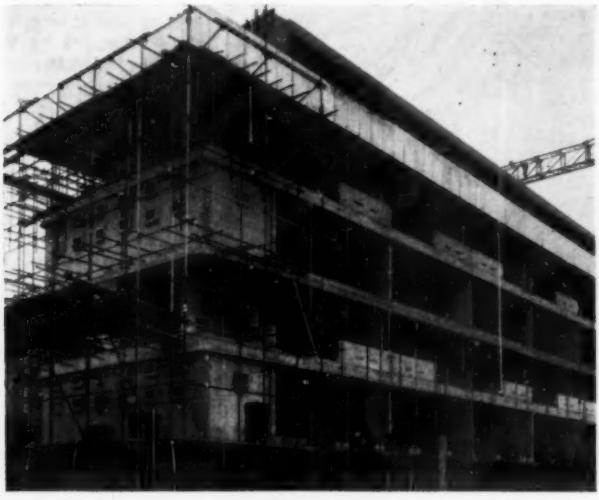


Fig. 2.

Factory near Newcastle upon Tyne.

A FACTORY recently completed at Blaydon, near Newcastle upon Tyne, for Churchill Gear Machines, Ltd., is built almost entirely of reinforced concrete and glass (*Figs.* 1 and 2). The single-story building is 220 ft. wide by 430 ft. long, and the width is divided into three bays of 52 ft. 6 in. each and one of 60 ft. A plan and sections are given in *Figs.* 3 to 5.

Overhead electric crane tracks extend the full length of two bays; the rails are 20 ft. above floor level and the clear height under the roof is 27 ft. 9 in. In each of these bays one crane has a capacity of 15 tons and the other of 5 tons. In another bay tracks for an overhead crane of 2 tons capacity extend for a length of 50 ft.; this crane is used for unloading steel billets and bars and placing them into storage racks, and a roadway extends into the building at one end for this purpose. The clear height of this bay and of the remaining bay is 16 ft.

All the crane-beams are 2 ft. 6 in. deep by 1 ft. 3 in. wide and are continuous over seven spans of 30 ft. 6½ in.; these beams are carried on brackets projecting from the columns (*Fig.* 6).

At the change in roof level clerestory windows are provided along the centre of the building.

The outer walls are of brick to 5 ft. above floor level, above which they are glazed. Direct light is diffused by rough-cast glass, and additional natural lighting to the central part of the floor is provided by twenty glass-and-concrete lights each 32 ft. by 6 ft.

The "Shell" Roof.

The "shell" roof has a thickness of 2½ in., increasing to 4 in. at the edges of each vault. The radius of curvature is 25 ft. and the chord-width 30 ft. 6½ in. The reinforcement consists of steel fabric and work-hardened bars arranged as shown in *Fig.* 7. The valley-beams and edge-beams are 1 ft. 6 in. deep by 1 ft. wide and project beneath the slab. The end and intermediate frames are 2 ft. deep

Fig. 1.

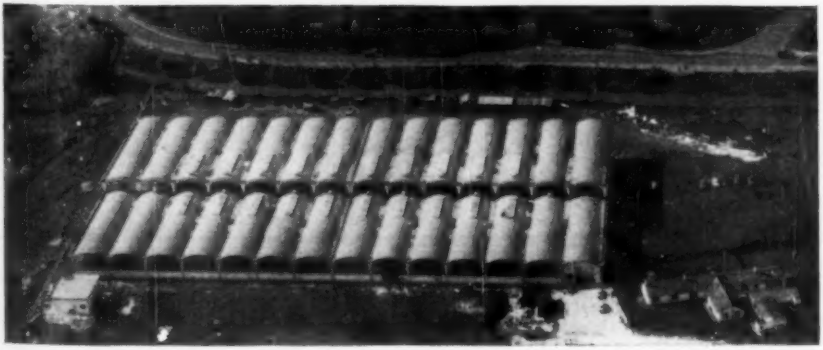


Fig. 2.

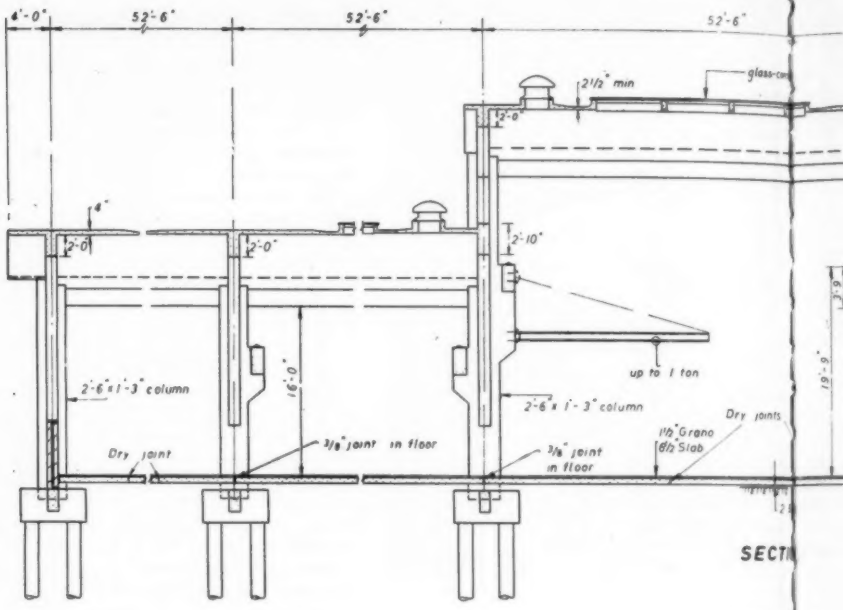


Fig. 3. Fig.

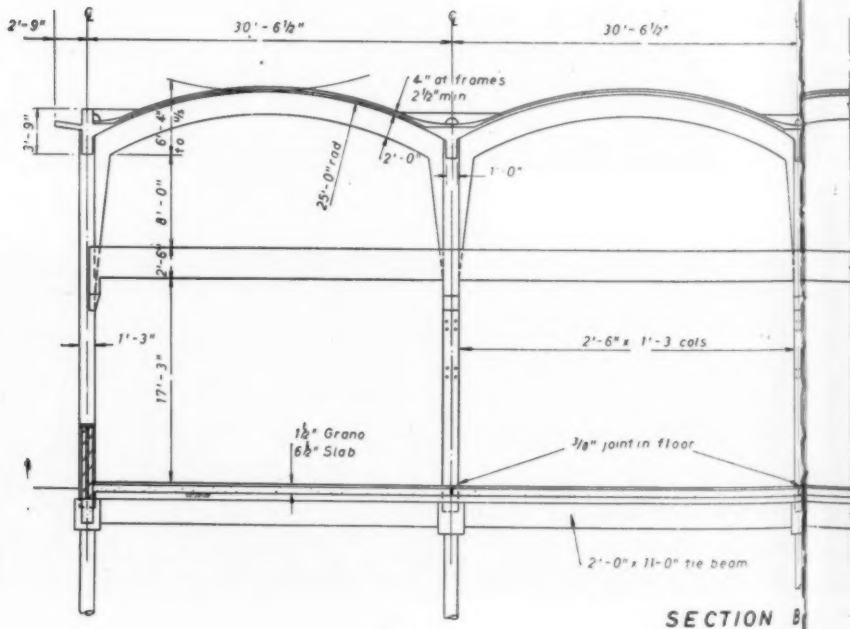
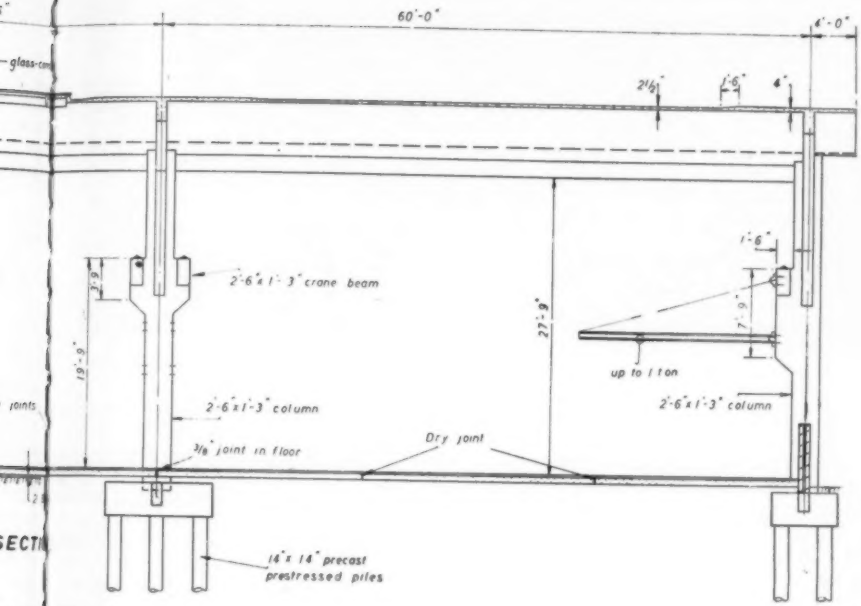


Fig. 4. Fig.



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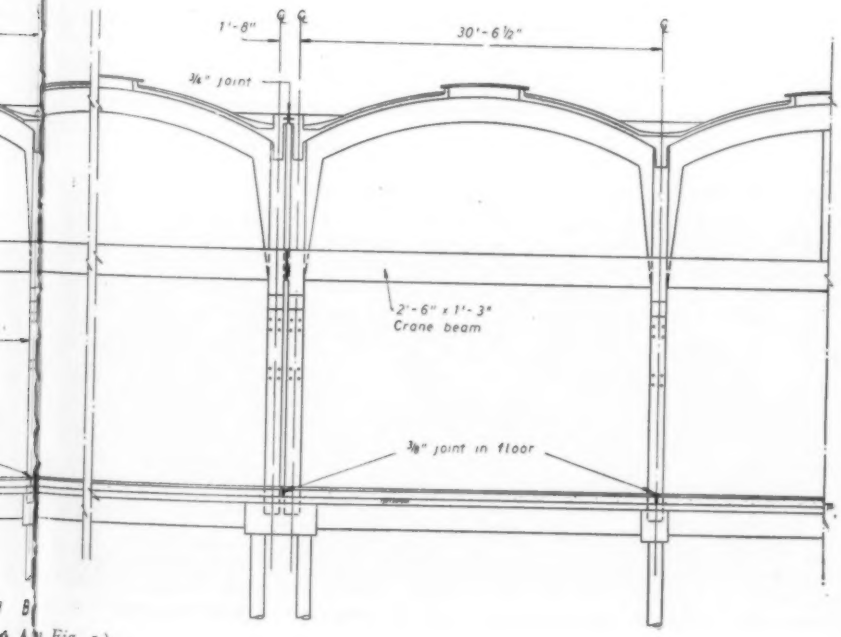


g. 4. (see Fig. 5.)

by 1 ft. wide and are curved to the radius of the shell. In view of the slenderness of these curved members, tapering fins are provided to strengthen the junction with the 2-ft. 6-in. by 1-ft. 3-in. columns which have their longer dimension in the other direction to resist the stresses produced by the cranes.

The columns are pin-jointed at the bottom and fixed at the top into the valley-beams of the roof. A series of very stiff frames is thus formed, and the bending moments in the columns are the same above and below the crane-brackets; the cross section of the columns is therefore

depths of 45 ft. to 50 ft. below floor level, but as piling progressed more variation was found and some of the piles are 60 ft. long.

All the piles were factory-made and prestressed. Most of them are 1 ft. 2 in. square and the remainder 1 ft. square, and they were used in groups of two or three to form foundations for the columns. The pile caps are connected by tie-beams which support the wall around the perimeter, additional piles being used to reduce the span where necessary. The maximum load on the larger piles is 60 tons and on the smaller piles 36 tons.

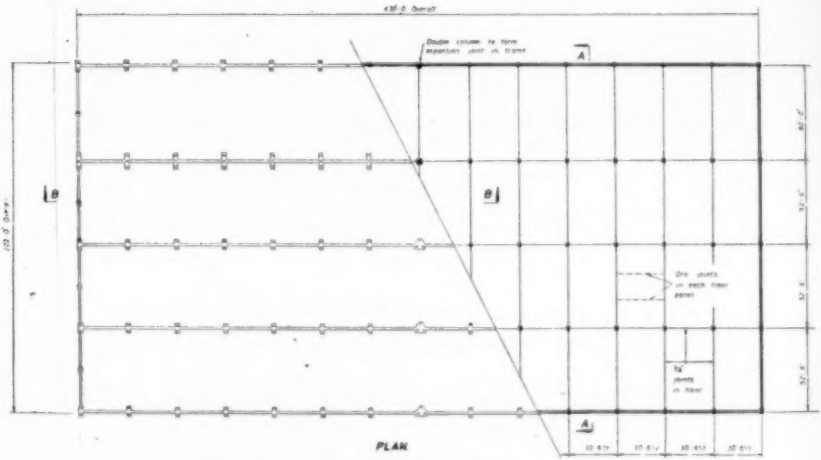


Fig. 5. (*Sections are given in Figs.* 3 *and* 4.)

constant. Jib-cranes with a capacity of 1 ton are attached to the columns (*Fig.* 6).

Advantages claimed for the design are that the stiffness of the frames maintains the correct distance between the crane tracks, that no large bending moments have to be resisted by the foundations, and that the full analysis of the structure can be delayed until a later stage of the work; in the case of this structure the foundations were well on the way to completion before a final decision was taken about the superstructure, cranes, etc.

Foundations.

The site consists of ash filling of variable depth overlying clay and gravel. Boreholes and test piles indicated that a satisfactory bearing could be expected at

An area of the ground floor measuring 52 ft. 6 in. by 275 ft. is carried on piles and designed as a suspended flat slab to carry machines that are very sensitive to differential settlement. The remainder of the ground floor rests on the filling material, which was compacted by a 10-tons road roller and covered with a compacted layer of colliery shale. The thickness of the suspended slab varies according to the load to be carried. The slab on the filling is 8 in. thick and has a topping of granolithic 1½ in. thick placed while the base concrete was green. A double layer of steel fabric is incorporated in all the ground-floor slabs.

The 250 piles were driven in eight weeks, using a rail-mounted piling frame equipped with a single-acting steam-hammer of

Fig. 6.—Showing Crane-beams on Brackets and Cranes Fixed to Columns.

3 tons weight and 4 ft. 6 in. stroke. The piling contract also included levelling and rolling the site and casting the pile-caps and tie-beams. The heads of the pre-stressed piles were stripped and the pre-stressing wires fanned out and bonded into the pile-caps. As the columns are pin-jointed at the foundations only simple recesses in the pile-cap were necessary.

Construction.

A tower crane (*Fig.* 8) was used in the construction of the superstructure, and the design of the roof was modified to permit the most economical use of this plant. The jib was long enough to cover the whole of a span of 52 ft. 6 in. when the crane-track was in the next bay. The first span was therefore designed to be self-supporting and the reinforcement which projected to connect with the next span was kept as short as possible. The crane-track had to be moved only twice, the fourth span being started when three or four bays of the third span had been constructed.

The columns were cast to their full height, a form of tremie being used inside the reinforcement. A very good finish was obtained on the columns.

The superstructure was built in about nine months. All the concrete was placed in six months, an average of more than two vaults per week. During the

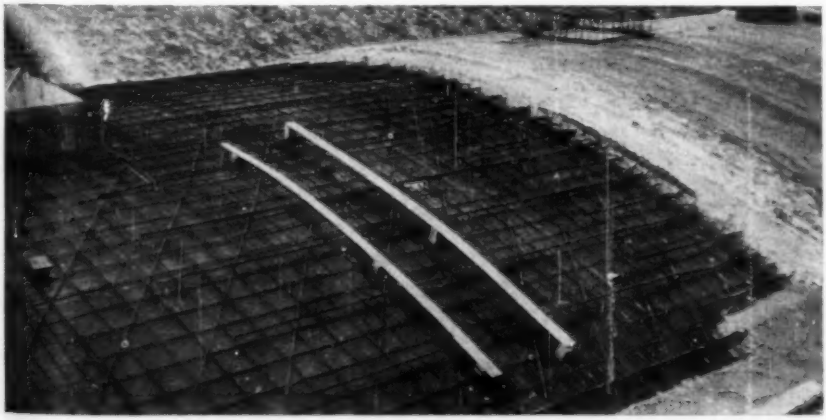


Fig. 7.—Arrangement of Reinforcement in Roof.

winter concreting proceeded with little interruption. Freshly-placed concrete was covered with tarpaulins and space-heaters burning propane gas were used, after which the concrete was covered with straw mattresses during the curing period.

The concrete was specified to have the following compressive strengths at 28 days: Columns, crane-beams, and tie-frames, 4500 lb. per square inch; roof, 3750 lb. per square inch; other work, 3000 lb. per square inch.

Mr. J. M. Angus, L.R.I.B.A., of Newcastle upon Tyne, was the architect. The contractors for the preparation of the site and the piling (including the test piles) were Dowsett Engineering Construction, Ltd. The contractors for the superstructure and finishing works were Messrs. Mowlem (Scotland), Ltd. The reinforced concrete work was designed and the reinforcement supplied by the British Reinforced Concrete Engineering Co., Ltd.

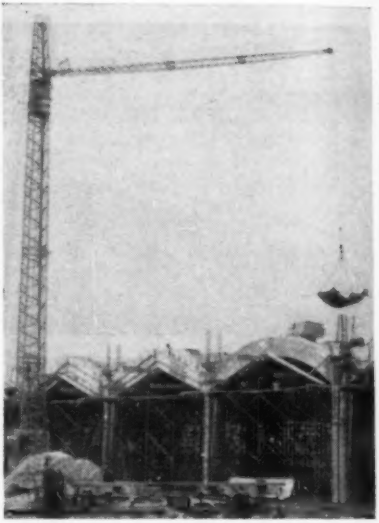
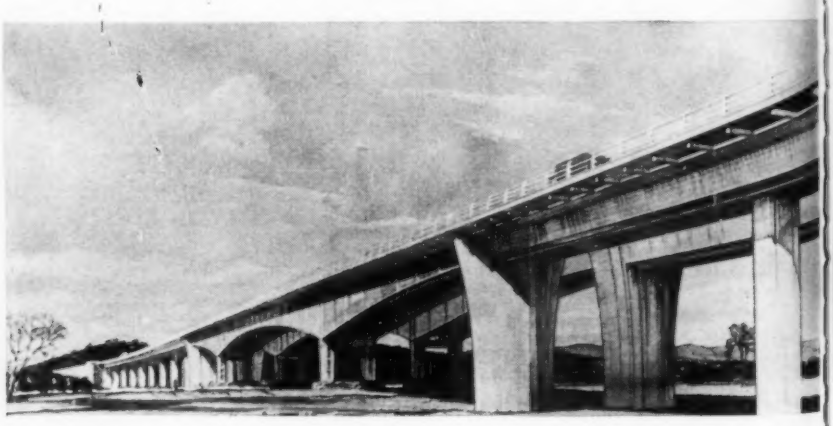


Fig. 8.—View during Construction.

A Bridge in Worcestershire.

THE illustration is of Queenhill bridge now being built to carry the Ross spur motor-road over the River Severn. The bridge has a total length of 2468 ft., comprising twenty-four spans each 82 ft. long in reinforced concrete and three spans over the river with spans of 131 ft., 238 ft., and 131 ft. in steel. The consulting engineers are Sir Alexander Gibb & Partners. The main contractors are Messrs. A. E. Farr, Ltd., and the steel spans are being built by Messrs. Head, Wrightson, Teesdale, Ltd.



A Reservoir in Monmouthshire.

PRECAUTIONS AGAINST SUBSIDENCE.

A SERVICE reservoir with a capacity of 2,000,000 gallons now being built at Gwernau, Monmouthshire, for the Rhymney Valley Water Board, is on a site that will be subjected to moderate subsidence due to mining. For this reason it is being built in separate compartments of equal size (*Fig.* 1), each with its own piping and instruments, so that one half may remain in service when the other is being repaired. Each compartment is 100 ft. square, and the depth of water will be 16 ft.

To provide for some flexibility each compartment is divided into nine parts by two joints in each direction which pass through the floor (*Figs.* 1 and 2), walls, and roof. In the floor the joints are 6 in. wide and in the walls they are 3 in. wide. The floors are 8 in. thick, and under these

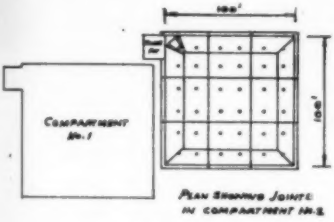


Fig. 1.

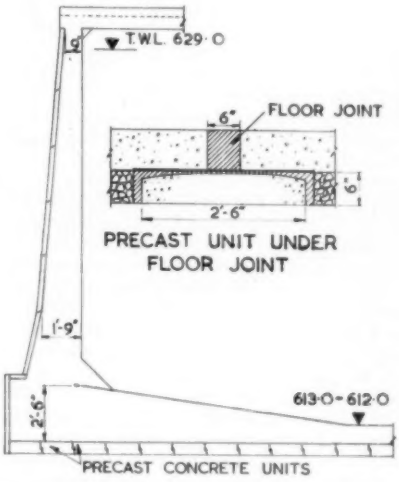


Fig. 3.—Section through Joint in Wall.

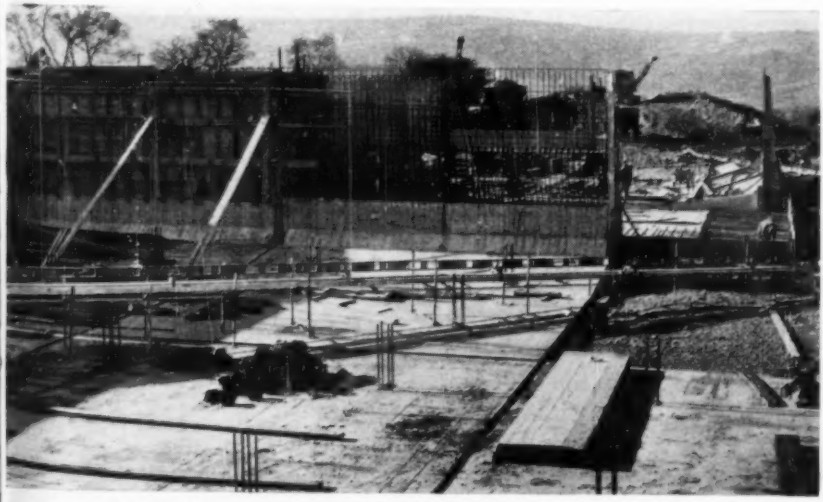


Fig. 2.—View during Construction showing Joints in Floor.

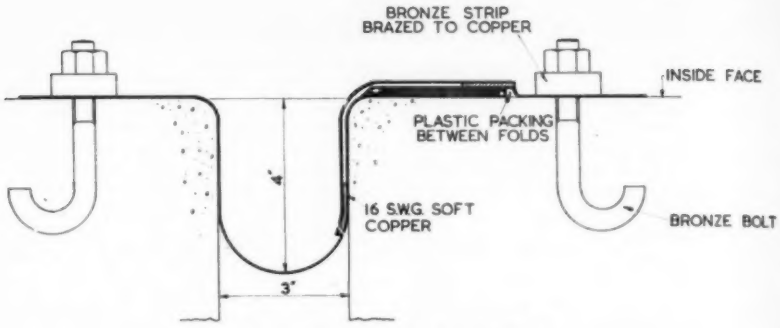


Fig. 4.—Detail of Joint in Walls.

joints there are precast concrete slabs (*Fig.* 3) covered with bituminous sheeting, and the gap between the slabs is filled with a bituminous rubber compound. The joints in the walls (*Fig.* 4) are 3 in. wide, and are closed with two sheets of copper each bolted to the inner face of the wall on either side of the gap. One of the sheets is bent to a U shape in the gap, and at their extremities the sheets are folded and interleaved with each other, thus forming a sliding joint. Over the sliding portion is a sheet of rubber-bitumen material 7 in. wide, sealed at both sides with an asbestos-bitumen compound. This sheeting can be easily replaced if it is damaged due to movement at the joint.

The wall (*Fig.* 3) is 1 ft. 9 in. thick at the bottom, tapering to 9 in. at the top.

The roof is of reinforced concrete and is divided into areas 33 ft. square each of which is designed as a flat slab supported on four columns; under the central panel the columns are braced (*Fig.* 5). The reservoir is embanked and covered with 1 ft. of earth. Drainage is provided under the floor and in the soil on the roof.

The engineer of the Rhymney Valley Water Board is Mr. W. H. James, B.Sc., A.M.I.C.E., and Mr. Maurice Nachshen, B.Sc., M.I.C.E., is the consulting engineer. The contractors were Messrs J. E. Jones & Co., Ltd.

Fig. 5.—Interior View.

Hangars and Workshop at London Airport.

BRITISH European Airways are adding new buildings at London Airport which will enclose an area of over 500,000 sq. ft. and consist essentially of two rows of five hangars, each 900 ft. long by 142 ft. wide, separated by a central workshop building 900 ft. long by 200 ft. wide, and with another workshop at one end. A basement extends across the workshop at the end and under the central workshop. The site comprised brick earth averaging 4 ft. deep over a bed of gravel 14 ft. deep over blue clay. The brick earth was replaced by hoggin compacted in 9-in. layers by a vibro-tamper, and densities of up to 140 lb. per cubic foot were obtained.

The Basement.

The basement beneath the central workshop is 820 ft. long, 70 ft. wide, and 20 ft. deep, and was excavated inside three cofferdams formed of sheet steel piles 28 ft. long and tied back to anchor-piles 50 ft. behind the face. Up to 30 piles were driven per day. The walls of the basement are waterproofed with three coats of asphalt, which is protected from the action of petrol (which lies on top of the ground-water) by three coats of plastic paint. Flotation of the basement is prevented by a layer of gravel beneath the floor, from which water entering the cut-off trench is piped to an ejector pit and pumped away.

The internal structure and roof were designed to achieve rapid construction. Columns are of standard sizes and cast in place to mezzanine level, and then to roof level after placing the precast mezzanine floor. All floor beams were precast to the underside of the slabs, which comprise prestressed precast units with infilling concrete tiles and a topping cast in place. The central line of columns extend through the workshop floor at 60 ft. intervals to support the roof. Access to the lift was made possible by using frames of up to 45 ft. span in place of the standard 15-ft. column spacing. Staircases generally were precast as complete flights.

The Hangars.

Each hangar comprises five bays 180 ft. by 142 ft. which are arranged to provide

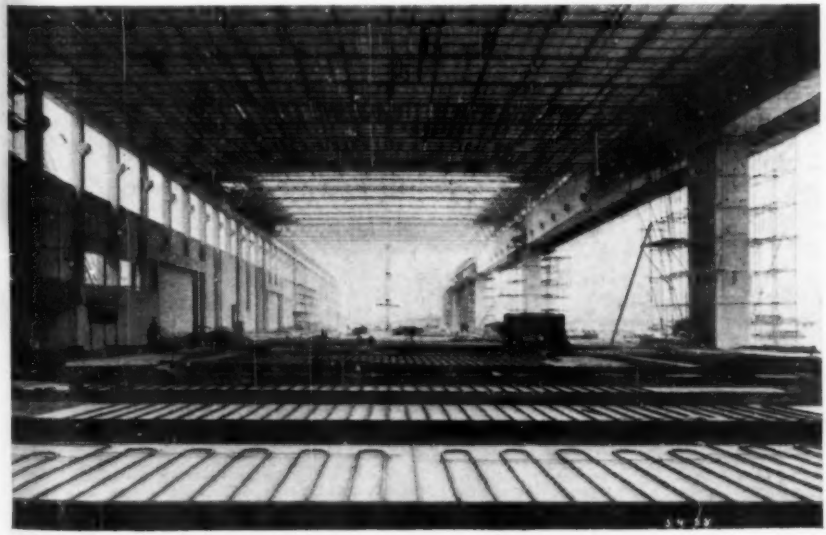


Fig. 1.—Prestressed Beams of 170 ft. Span to Right.

a clear floor area of 900 ft. by 135 ft. with five access doors with clear spans of 168 ft. On each side of the door openings are columns 6 ft. 6 in. by 3 ft. at 171 ft. centres with one column having wings 9 in. thick to make it very rigid, thus providing an overall area of 6 ft. 6 in. by 12 ft. (*Fig.* 2). A hollow prestressed beam 14 ft. deep by 6 ft. 6 in. wide with sides 5 in. thick, a top slab 8½ in. thick, and a bottom slab 5½ in. thick spans between these columns. The sides of the beams are connected by precast members at 10 ft. centres, half of which have a corbel to support a crane rail on the inner face and a projection on the outer face to carry a canopy and the door head.

The main beam is prestressed with thirty-eight Freyssinet cables each containing twelve 0·276-in. wires which were passed through steel ducts cast in the sides and soffit before the concrete was cast. These cables were tensioned to 73 tons per square inch by jacking from both ends, and later grouted with cement-sand grout with a water-cement ratio of 0·45 and an expansive chemical.

A compressive strength of 6000 lb. per square inch at 28 days was specified for these beams, and this was obtained with a mixture of 112 lb. of cement, 115 lb. of sand, 154 lb. of ¾-in. to ⅜-in. shingle, and 90 lb. of ⅜-in. to $\frac{3}{16}$-in. shingle, and a water-cement ratio of 0·37. A typical result of tests of the compressive strength of this concrete at 28 days was as follows (in lb. per square inch): Highest, 8850; average, 7850; variation from average, 2250; standard deviation, 554; coefficient of variation, 7 per cent.

The end walls are largely glazed, the lower and side portions being of reinforced concrete 6 in. thick. The precast columns are fixed at the bases, and the beam at eaves level is so designed that the purlins can slide when the wall moves due to wind pressure or as the roof expands. The columns are designed to reduce horizontal deflection rather than to carry direct load. These columns are hollow and weigh about 17 tons. The bases were provided with a tapered socket into which the tapered end of the column was concreted.

The rear wall consists of five bays of hollow precast columns and beams corresponding to the five door openings. The columns are on bases which also carry the

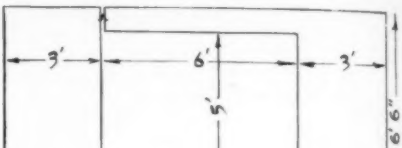


Fig. 2.—Cross Section through Main Columns.

adjacent precast columns for the workshop. In this case tapered socket-bases could not be employed; instead the columns were precast with the lower reinforcement protruding, and with an inner lattice core and base-plate capable of supporting the weight of the column. To carry this column a base 8 in. thick was cast containing the main foundation reinforcement. The column was then put into position, stabilised with guys, and the top lift of the base, approximately 3 ft. 6 in. deep, was concreted. This was simpler and cheaper than the socket-type base.

The columns are at 20 ft. centres except at the centre of the bay where a clear opening 40 ft. wide is provided in case the hangar is used for longer aircraft which might project through the rear wall. This was achieved by carrying two of the columns on a beam 28 ft. above floor level, spanning 42 ft. to two extra columns. This beam was constructed in three longitudinal parts, the two outer parts being precast and the central one cast in place. In order to avoid congestion of reinforcement the two upper columns were each prestressed with the beam by means of six cables each containing 12 wires of 0·276 in. diameter tensioned by the Gifford-Udall system. The beam was designed to transmit the wind load as well as the vertical loads to the extra supporting columns on each side of the opening.

The columns generally are connected by precast beams at the bottom of the glazing, about 33 ft. above the floor, and by a hollow beam above the glazing. There is a flint-lime brick wall to a height of 32 ft. between the columns. As these columns are 65 ft. long and weigh 18 tons, a maximum tolerance of ¾ in. at the top

was permitted in any direction, and all the attachments to the column, including the glazing, were designed to allow for this tolerance.

The roof consists of welded tubular frames sprayed with 0·005 in. of aluminium. They are 7 ft. overall in depth, span 133 ft., are at 20 ft. centres, and weigh only 6 tons. Between the frames are monitors of joist section, also metal-sprayed. The roof covering is patent glazing over the monitors, and aluminium decking between and around the monitors.

Precast concrete was used extensively to complete the walls at roof level. On the main door beam a precast fascia is added to the end of the outrigger. Hollow precast beams span between the outriggers to provide the canopy over the folding doors. Beneath the canopy an aluminium-faced asbestos ceiling is provided against which the top of the door presses to exclude draughts.

Above the main beam are precast transverse frames between which precast purlins are bolted to carry a precast parapet and corrugated asbestos-cement sheets. The gutters are of pressed steel covered with bituminous felt.

At the top of the rear wall a hollow beam is provided below the level of the steel girder. Precast ribbed channel-shaped units were placed on this beam, and a precast combined gutter and coping is bolted to the top of this member. Throughout the top fascias strips of mastic are used to protect the vertical joints.

The Workshop.

The central workshop is 200 ft. wide and is roofed by north-light welded tubular frames of 100 ft. by 60 ft. span. These are designed to carry two 5-tons loads on any frame at 20 ft. centres to allow for the installation of small cranes or monorail transporters.

The end bays only are braced at the level of the lower beam. To allow the central gutter to be used as a walkway and to avoid the use of a beam between the central row of columns, which are at 60 ft. centres, a lattice member made up of the ends of the hipped frames with top booms added outside the cladding is used. This provides a complete roof of tubular lattices.

In the case of the central row of columns reinforcement bars project from the basement structure and, to allow the columns to be precast, a lattice foot was provided as in the case of the columns to the hangars, and only the bottom 4 ft. of the column were cast in place. The

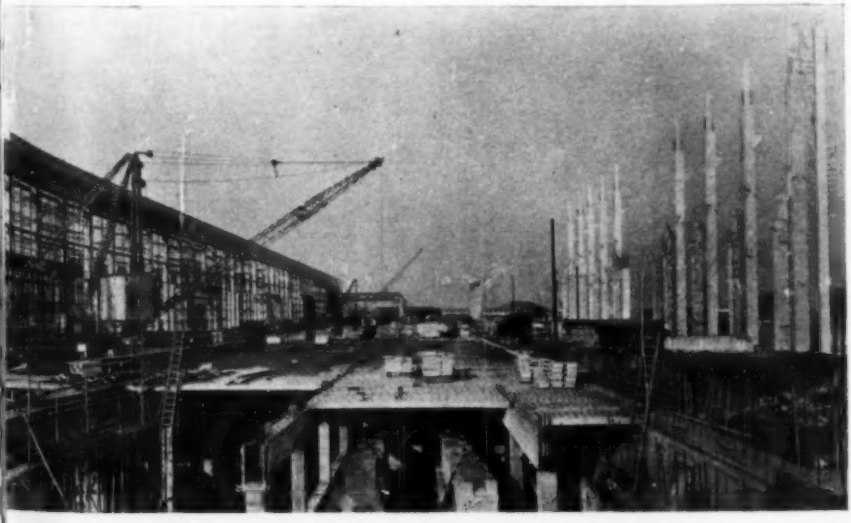


Fig. 3.—Construction of Basement under Workshop.

lower part of the column was wedge-shaped to allow air to escape while the bottoms were concreted in, and a box was constructed at two faces of the column to ensure complete compaction. This led to a small corbel being cast, and this was cut off before it hardened.

The north wall of the workshop comprises precast columns with precast purlins at 8 ft. centres vertically. The wall was clad with patent sheeting externally and asbestos-faced panels internally.

The End Workshop.

The workshop extending the full width of the building at one end is also a combination of a tubular steel roof and concrete supporting members. On the side adjacent to the hangars and the central workshop, precast columns are used at 50 ft. centres and capped by precast beams. A similar arrangement, with columns at 16 ft. 8 in. centres, forms the outer face of the building. The tubular welded steel roof is a series of monitors at centres of 16 ft. 8 in. so that the roof load is transmitted directly to the outer columns and as concentrated loads to the beams of 50 ft. span where this workshop adjoins the main building. All the beams are hollow near the support so that a connection can be cast in place over the column.

Ground Slabs.

A concrete ground slab is provided in front of each hangar. On one side the new slab has an average width of 150 ft. and with the adjacent slab of the existing base provides a complete width of 425 ft. On the other side the slab is 300 ft. wide.

The slabs are 10 in. thick and were cast in strips 15 ft. wide on building paper. The distance between expansion joints varies up to 220 ft. Contraction joints were formed by use of a vibrating blade to one-third of the depth of the slab at 15 ft. centres. Strips of hardboard $\frac{1}{4}$ in. thick were placed in the joints and the top $\frac{3}{4}$ in. was later cut out and the space filled with a bituminous compound. The longitudinal joints are plain butt joints painted with bitumen.

The concrete was specified to have a compressive strength of 5000 lb. per square inch at 28 days when tested in 6-in. cubes. A mixture of 112 lb. of cement, 184 lb. of sand, 326 lb. of $1\frac{1}{2}$-in. to $\frac{3}{4}$-in. aggregate, 92 lb. of $\frac{3}{4}$-in. to $\frac{3}{8}$-in. aggregate, and 106 lb. of $\frac{3}{8}$-in. to $\frac{1}{10}$-in. aggregate and a water-cement ratio of 0·48 were used, and 0·28 lb. of carbon black was added to darken the surface.

Floors.

The floor of the hangar was cast in two

Fig. 4.—Erecting Precast Members.

stages. On a bottom slab of 1 : 6 concrete were laid the steel tubes for the floor-heating system followed by 10½ in. of similar concrete and a granolithic topping tamped into the structural concrete by machine. The bays vary in width from 12 ft. to 15 ft., and joints were formed by a batten at the bottom of the slab and a saw-cut immediately above a few days after laying.

The floor of the workshop consists of a bottom slab 6 in. thick and a 4-in. top slab in which are embedded electrical conduits and other services, with a granolithic topping.

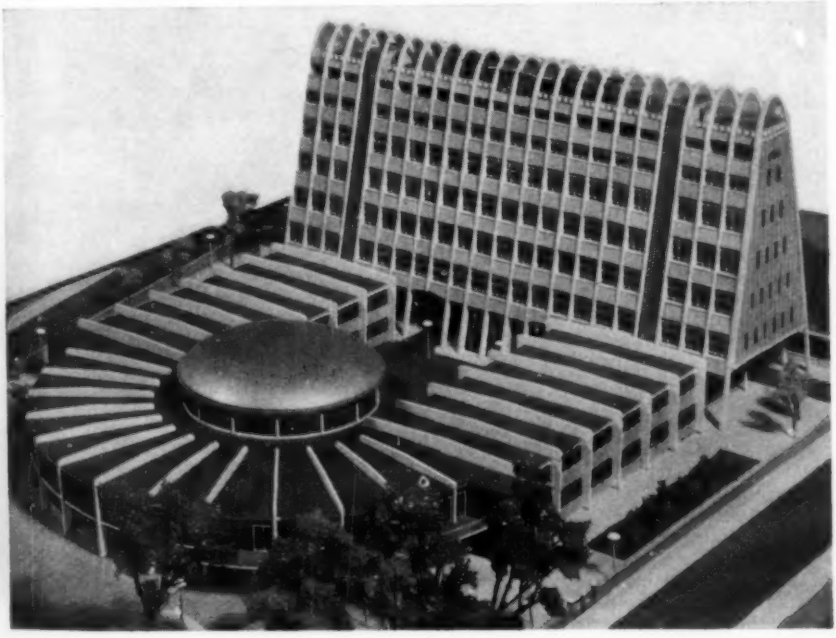
The first two bays of the hangar, 360 ft. long, were completed in October 1958 and the work is to be completed early in 1960. The consultants are Messrs. Scott & Wilson, Kirkpatrick and Partners. The architects are Messrs. Ramsey, Murray, White & Ward. The contractors are Messrs. Holland & Hannen and Cubitts (Great Britain), Ltd., and the precast members were made by The Concrete Development Co., Ltd.

School at Manchester.

THE Domestic and Trades College now being built at Fallowfield, Manchester, comprises a seven-story building 120 ft. high with arch frames at 10 ft. centres, and a horseshoe-shaped two-story building with rectangular frames at 20 ft. centres and a span of 60 ft. Also adjoining are a tailoring block and a gymnasium.

The multiple-story building is 72 ft. 4 in. wide at ground level and 23 ft. 7 in. wide at the seventh floor. It is founded on continuous strip footings on stiff brown clay, and the ground floor is open for use as a service road. The frames are 36 in. deep by 15 in. wide ; rainwater-pipes are cast within the columns, and all other services are in a central vertical duct.

The circular assembly hall is about 82 ft. in diameter. The ground floor is intended for use as a car park, and the



floor of the hall above is supported at the centre on a mushroom-headed column tapering from 3 ft. 6 in. in diameter at ground level to 5 ft. 1 in. at the springing of the flared top; the diameter of the flared head which forms part of the floor is 25 ft. and its average thickness is 3 ft. The roof of the dome is of stressed aluminium.

The architect is Mr. Leonard C. Howitt, the City Architect. The consulting structural engineers are Messrs. L. G. Mouchel & Partners, and the contractors Messrs. J. Gerrard & Sons, Ltd.

A Winder Tower in Northumberland.

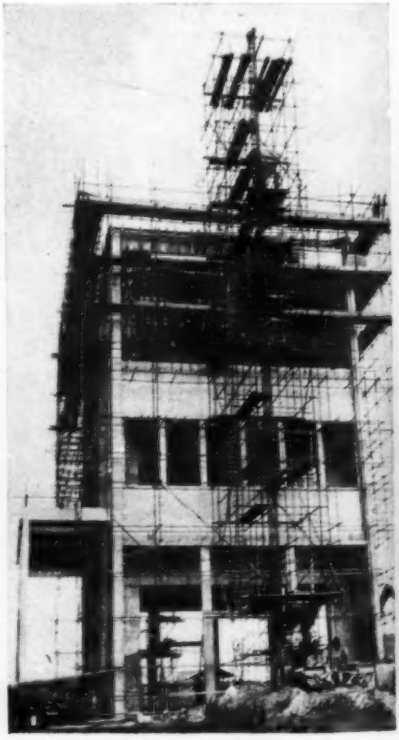
THE reconstruction of the surface works at the Rising Sun Colliery in Northumberland for the National Coal Board is now well-advanced. Several of the structures are in reinforced concrete, including a bunker of 1000 tons capacity and a winder tower over the downcast shaft for the ventilating system.

The winder tower, which is entirely in reinforced concrete, will accommodate two multiple-rope friction winders, each of 750 h.p. It measures 68 ft. by 42 ft. and the main roof is 110 ft. above ground level. There are three floors at heights of 28 ft., 56 ft., and 80 ft. above ground level; the first floor will be used for servicing the cages and ropes, the second floor will contain the machinery and switchgear associated with the winders, and the third floor will carry the two winders and an auxiliary winch for servicing the ropes and shaft equipment. The auxiliary winch is designed for a load of 22 tons acting over pulleys in a penthouse on the roof; the penthouse also contains the filtration equipment for the cooling-air.

The mine shaft is 22 ft. in diameter and is capped at ground level by a reinforced concrete "collar" 30 ft. square at the outer edges and around which 84 bored piles have been installed as the foundation for the four L-shaped columns of the tower. The floors are of beam-and-slab construction spanning between the four columns, so that the whole of each floor is unrestricted. The beams of the winder-floor are designed for a breaking-load on the ropes of 220 tons on each winder at the same time; this occurs in the centre of a floor 40 ft. square.

The tower will be totally enclosed by glazing with the exception of louvres between the first and second floors which provide most of the ventilation air entering the shaft. The cost of the structure is estimated to be £70,000.

The contractors for the whole of the civil engineering work in the surface reconstruction contract are Messrs. J. L. Kier & Co., Ltd., and the consulting engineers are Messrs. Posford Pavry & Partners.



Test of a Staircase.

THE reinforced concrete staircase shown in *Fig.* 1, comprising two flights and a half landing and spanning from the ground floor to the first floor without an intermediate support, was tested to ascertain the deflections caused by various arrangements of load.

Deflections were measured by seven dial-gauges, recording to one-thousandth of an inch, which were supported by a steel scaffold independent of the staircase. The position of the dial-gauges is shown in *Fig.* 2. The load was applied in sixteen stages by stacking bricks on the steps and landing, and the deflections were read from the dial-gauges at each stage.

The loading stages were as follows; the parts indicated by the reference letters are shown on the plan in the upper part *Fig.* 2.

(1) Half design load on D.
(2) Half design load on D and C.
(3) Design load on D and half design load on C.
(4) Design load on D and C.
(5) 1·25 design load on D and design load on C.
(6) 1·25 design load on D and C.
(7) As (6) plus half design load on B.
(8) As (7) plus half design load on A.
(9) As (6) plus design load on B and half design load on A.
(10) As (9) but with design load on A.
(11) As (6) but with 1·25 design load on B and design load on A.
(12) As (11) but with 1·25 design load on A.
(13) Load removed from A.
(14) Load removed from B.
(15) Load removed from C.
(16) Load removed from D.

The staircase was subjected to very severe conditions at the completion of stages 6 and 14, when only the upper flight and the half of the landing adjacent

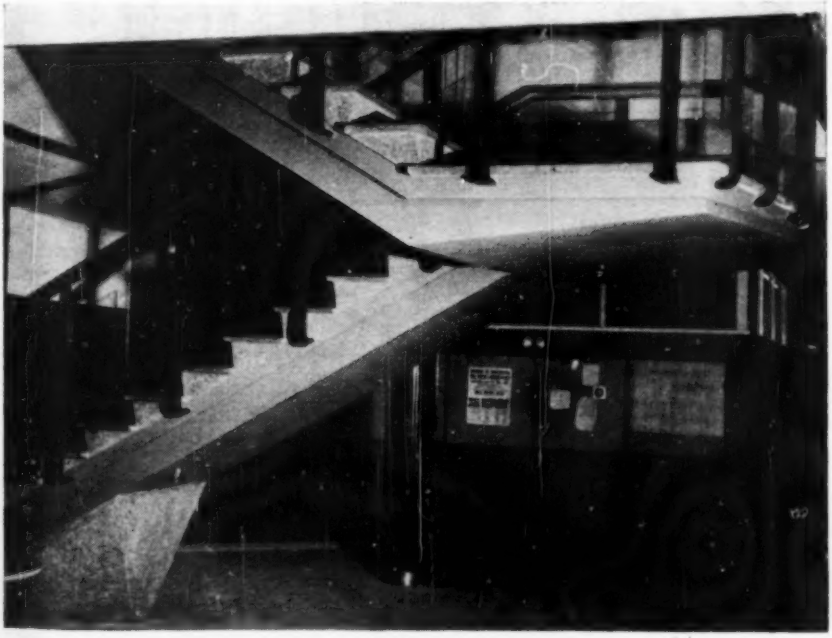


Fig. 1.

to the upper flight were loaded with the design load plus an overload of 25 per cent. and there was no load on the remainder.

Fig. 3 shows the deflections recorded at the outer corners of the landing and the horizontal movements observed at the outer corner of the landing on the same side as the lower flight. The direction of this horizontal movement during the application of the load was from the lower flight towards the upper flight.

The total load was left in position for sixteen hours between stages (12) and (13) and the creep was observed. The average increase in deflection during this time on all seven instruments was 11 per cent. When all the load was removed the average recovery of all seven instruments was 73·4 per cent. immediately, and 79·6 per cent. after 1¾ hours.

The maximum vertical deflection recorded was 0·082 in. at the outer corner of the landing on the same side as the lower flight, and the maximum horizontal movement of the landing was 0·052 in. These deflections had decreased by 80 per cent. and 71 per cent. respectively 1¾ hours after the removal of the load.

The staircase is at the Scunthorpe Riddings secondary modern school built for the Lindsey County Council, in which

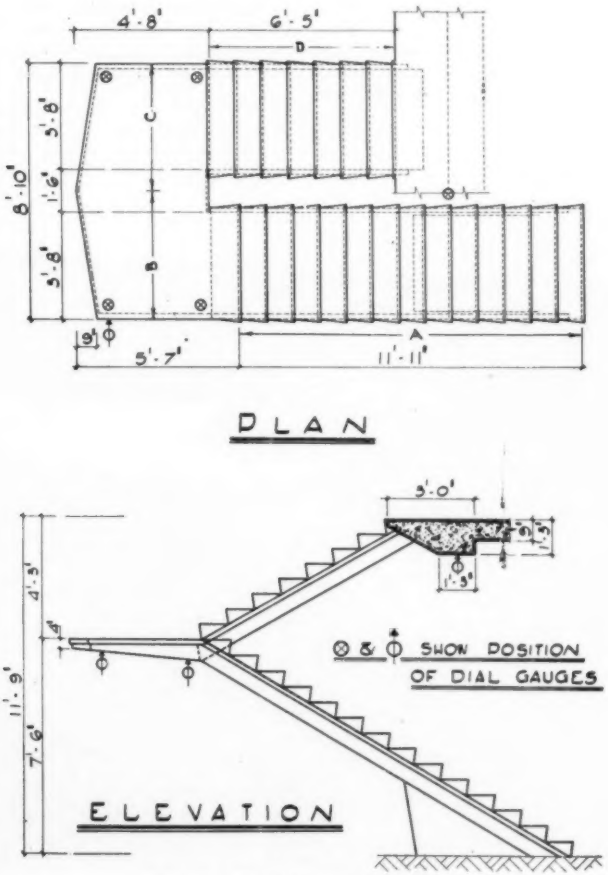


Fig. 2.—Positions of Gauges.

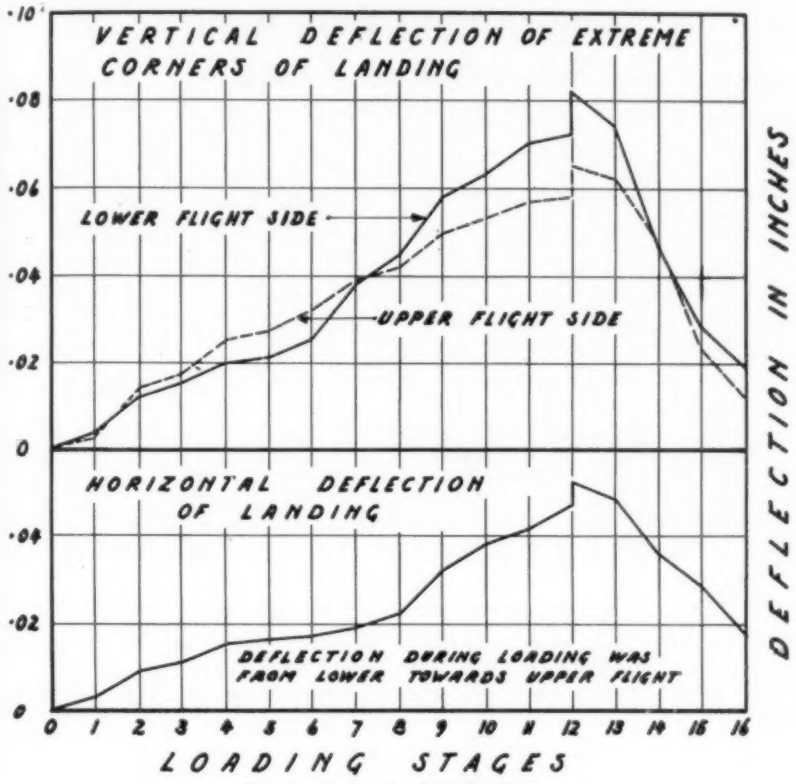


Fig. 3.—Record of Deflections.

there are three such staircases. The architects are Messrs. Denis Clarke Hall and H. S. Scorer, the consulting engineer is Dr. K. Hajnal-Kónyi, and the contractors are Messrs. F. Shepherd & Son, Ltd.

Scientific Training in the U.S.A.

THE U.S.A. Engineering and Scientific Manpower Commissions state that at present about 8500 students at U.S.A. universities are awarded Ph.D. degrees in scientific and engineering subjects each year, and that in ten years' time the number will be about doubled. No reason is given for this estimate. It is, however, stated that " emphasis is being placed on subsidising students who are already swamping institutions ill-equipped and ill-staffed to handle large numbers, rather than on the number and the quality of the teachers and the proper equipment of the colleges ". This suggests that the expected increase in the number of doctorates awarded may be a result of the lowering of the standard of knowledge required for the award of Ph.D. degrees.

It may be recalled that in Great Britain £100,000,000 is being spent on new buildings and equipment at colleges.

North-light Shell Roof Supported on Frames.

An unusual feature of the reinforced concrete bus depot now under construction for the Caerphilly Urban District Council is the use of frames to support a north-light roof of three bays spanning 67 ft. (*Figs.* 1 and 2), instead of the more usual construction in which the shells are supported on longitudinal beams carried on columns along the sides of the building. The roof is continuous over five spans, three with lengths of 32 ft. and two of 40 ft. The bays are each 22 ft. wide, so that the total area covered is 67 ft. by 176 ft. The clear internal height is 19 ft.

The roof is designed as spanning between the frames, using a simplified method of calculation* due to Mr. H. Tottenham. The main dimensions are shown on the cross sections; the shells are basically 2½ in. thick. The frames were designed by the strain-energy method as statically-indeterminate structures with curved members of variable cross section. This leads to five simultaneous equations, the solution of which gives the three unknown forces in the bottom member and the two compressive forces in the top member.

* "A Simplified Method of Design of Cylindrical Shell Roofs," by H. Tottenham. Journal of the Institution of Structural Engineers, June, 1954.

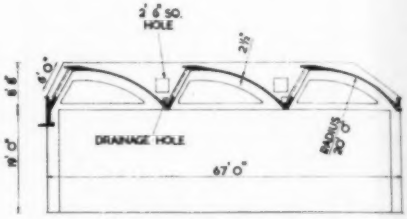


Fig. 1.—Cross Section.

To provide external access from one bay to the next, openings 2 ft. 6 in. square were left in the frames in order to avoid the necessity of climbing to the highest point to pass from one bay to the next for maintenance and window-cleaning. Holes 12 in. wide are left in the frames for drainage purposes along the valleys. (In the reinforcement details on the drawing shown in *Fig.* 2 the letter T indicates Tentor bars.)

The architects are Messrs. Jackson & Greenen. The structural design is by G.K.N. Reinforcements, Ltd., and the contractors are Messrs. E. Taylor & Co., Ltd.

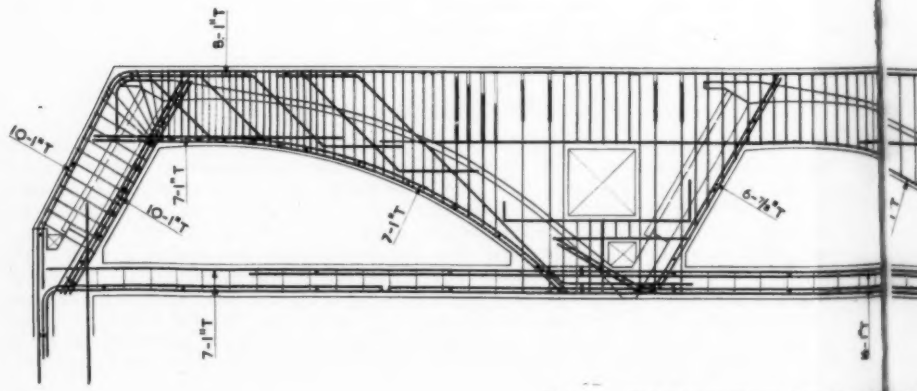
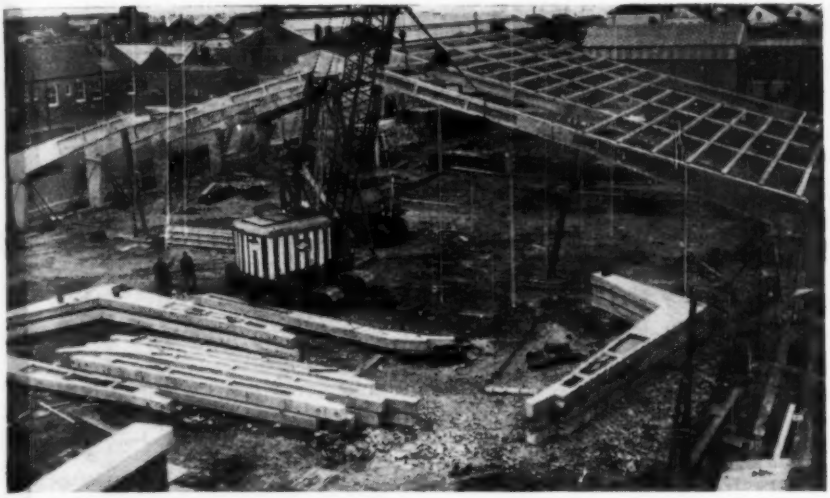
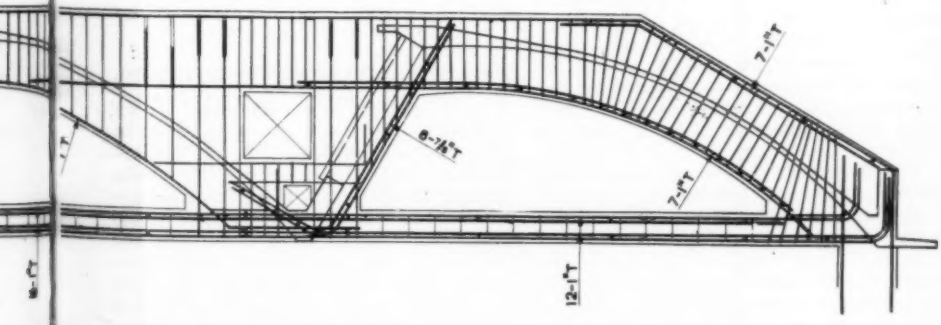


Fig. 2.—Reinforcement Details of N-li

Precast Frames of 132 ft. Span.



THE frames shown in the illustration in course of erection at Great Yarmouth have a span of 132 ft., which is thought to be the longest of its type in this country. Each frame comprises four parts, which were erected by crane and bolted together. The structure was designed by Messrs. R. E. Eagan, Ltd., who cast the frames on the site and erected them with the aid of a mobile crane.



-light Roof Supported on Frames.

A Building in Nigeria.

THE new head office of Barclays Bank D.C.O., Ltd., in Lagos is due to be completed this year. It is 200 ft. long, 68 ft. wide, and 80 ft. high, and is constructed in reinforced concrete throughout.

There will be nine floors, comprising a strongroom in the basement, a car park at ground level, offices on a mezzanine floor and on the third to sixth stories, and a canteen on the seventh floor (*Figs.* 1 and 2). There will be three passenger lifts, a lift giving access to the strongroom, and a service lift. The entire building will be air-conditioned except the car park on the ground floor, the longer walls of which will be in the form of a grillage of precast concrete members. The cooling tower and filters for the air-conditioning system will be on the roof. The heights from floor to ceiling are 7 ft. 9 in. in the car park on the ground floor, 19 ft. 3 in. in the banking hall, and 9 ft. 3 in. in the upper stories. There are two vertical joints in the superstructure.

At one end of the building lavatory accommodation is provided on each floor between the passenger lifts; this part of the building is of frame construction, and the rest of the building has flat-slab floors without flared heads to the columns or thickened panels over them, so that partitions may be erected where they are required or moved without restriction.

The site consists of fine to medium loose sand, and is within 70 yd. of the Lagos lagoon. The greatest permissible pressure on the sand allowed by the by-laws is ½ ton per square foot, and the building is supported on a semi-buoyant cellular raft which is to be used for storage of securities. The site is completely waterlogged, and a system of temporary drainage was designed by the consulting engineers in order to permit work below ground-level to be done in the dry.

The basement is waterproofed with several layers of bituminous emulsion reinforced with glass fibre. In addition to the fact that asphalters were not available and local labour could apply the emulsion, there are the advantages that the emulsion is less brittle than asphalt and the use of emulsion at the joints between the walls and floors avoided external projections which might be fractured and the waterproofing thereby impaired.

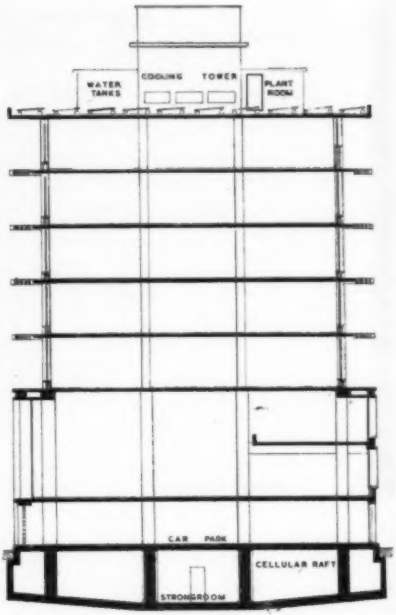


Fig. 1.—Cross Section.

The architects are Messrs. Walker, Harwood & Cranswick, the consulting engineers are Messrs. Maurice Nachshen & Partners, and the contractors are Messrs. Costain (West Africa), Ltd.

Data on Welded Mesh Reinforcement.

A NEW edition of the Maxweld Fabric Technical Data Book has been issued by Messrs. Richard Hill, Ltd., of Middlesbrough. It includes eight new tables giving the size of fabric required for five types of spans from 6 ft. to 14 ft. subjected to eight different loading conditions and with six thicknesses of slab. Information is also given on the use of the fabric for fire protection and for the encasement of structural steel. Copies of the book may be had on application.

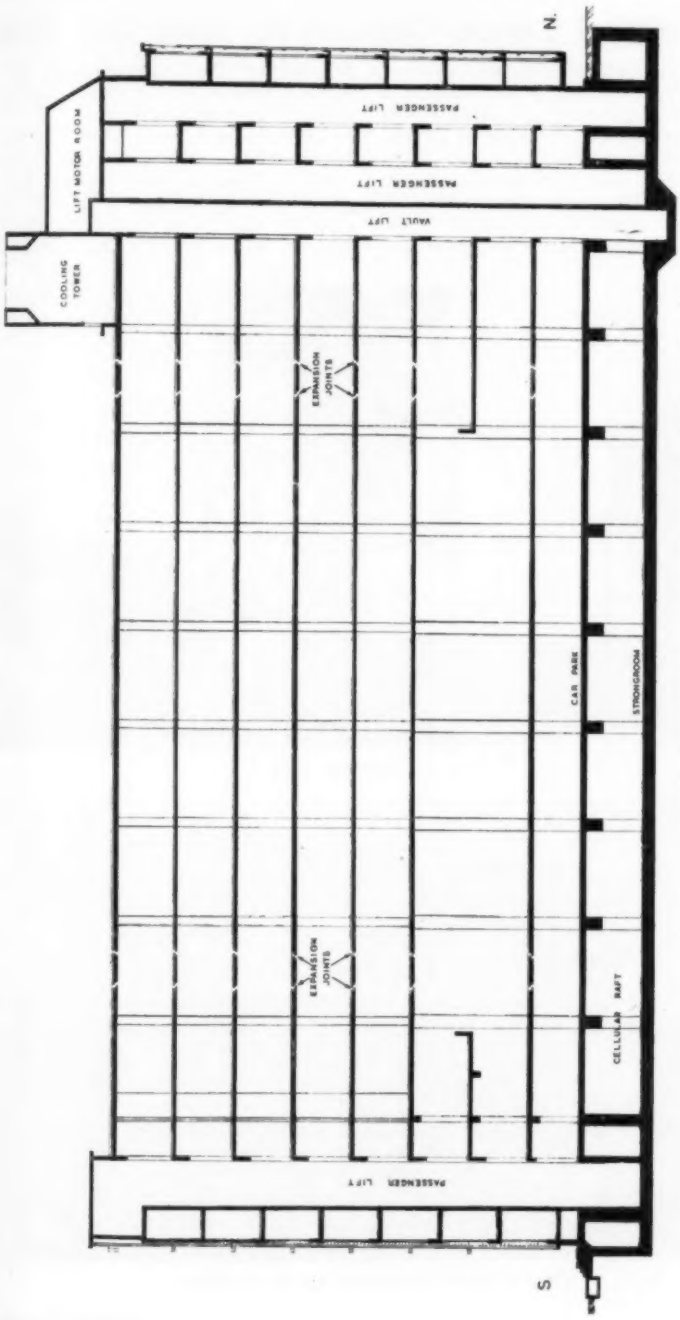


Fig. 2.—A Building in Nigeria : Longitudinal Section.

(See facing page.)

Delivery of Loose Cement by Rail and Road.

THE Scottish Region of British Railways, the contractors (Messrs. John Mowlem & Co., Ltd.) and the cement makers (Clyde Portland Cement Co., Ltd.) have collaborated in arrangements for supplying cement to the nuclear power station in course of construction at Hunterston. The silos, the weighbridge, and the compressor shown in *Fig.* 1 have been provided by British Railways at East Kilbride station, which is three miles from the site of the power station.

The objective is to meet the varying requirements of the work without disturbance to other railway traffic. Between 30,000 and 40,000 tons of cement will be used in the construction of the power station, and the daily requirements will

Fig. 1.—Storage Silos.

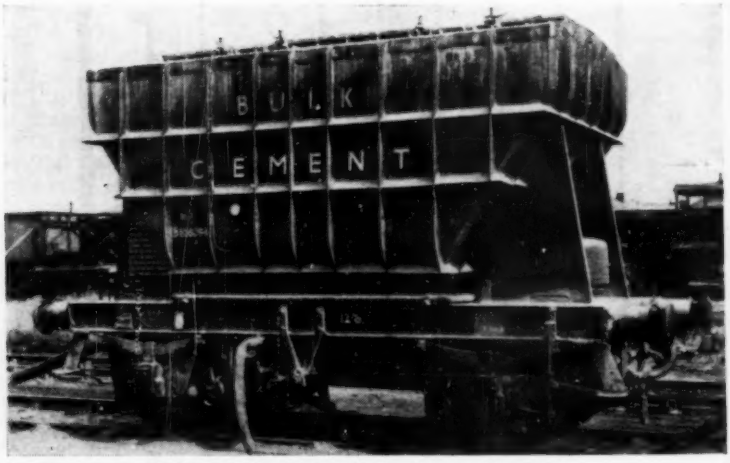


Fig. 2.—Rail Truck for Loose Cement.

depend on the weather and the stage of construction. Deliveries will also have to be made continuously to many different parts of the site, and road vehicles owned by the cement makers are used to take the cement from the railway station to the site, a distance of three miles.

The loose cement is loaded at the factory into air-discharge railway wagons (with a capacity of 90 tons) by means of a portable Diesel compressor-pump at the rate of one ton per minute, and the empty wagons are returned to the factory. A similar wagon with a capacity of 20 tons is shown in *Fig.* 2.

The road vehicles are loaded from the silos by gravity, the weight of cement in each vehicle being measured by means of an optical-projection weighbridge with a capacity of 28 tons, situated under one of the silos. At the site the vehicles are emptied by means of compressed air.

The silos are so designed that they can be dismantled and re-erected elsewhere when the power station has been completed.

A Bridge in Scotland.

THE bridge over the railway at Paddy's Rickle on the Carlisle–Glasgow trunk road A74 has been replaced by a structure comprising reinforced concrete abutments and a prestressed deck. The site is an exposed one near Beattock Summit, and the road carries heavy traffic.

The new bridge has an angle of skew of 71 deg. and, as the width of the new road and verges is 47 ft., the width of the bridge parallel to the railway is 266 ft. The span is 32 ft., and the headroom above rail level is 16 ft. The bridge has been designed for 45 units of Ministry of Transport abnormal (H.B.) loading. An important consideration in choosing the type of bridge was the need to interfere as little as possible with the use of the railway.

The abutments are parallel to the railway and are designed as propped cantilevers with mild steel reinforcement. Although rock is only 6 ft. below the ground surface, because of the presence of the railway lines it was decided to avoid the open excavation required for continuous foundations on the rock. Instead the abutments are on plain concrete piers 11 ft. by 3 ft. in plan at 12 ft. centres, and the

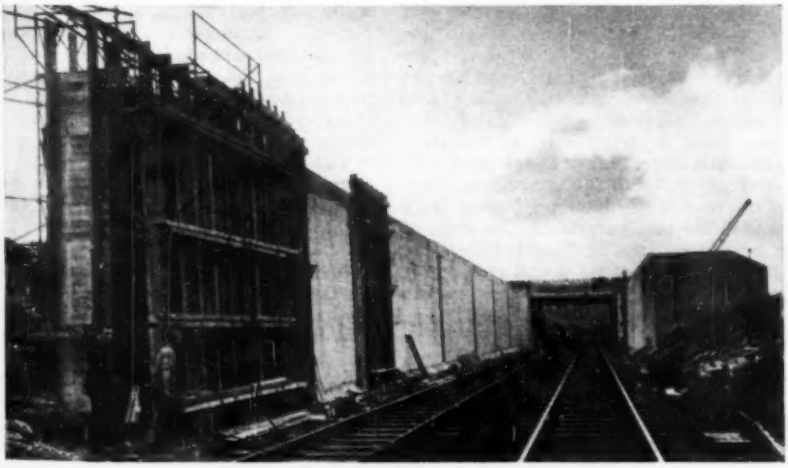


Fig. 1.—Bridge and Embankments.

Fig. 2.—Prestressing the Deck Transversely.

sequence of constructing the piers was closely controlled. The design allows for bearing pressures up to 16 tons per square foot.

The deck has been designed as a quasi-isotropic slab and consists of prestressed precast inverted T-beams 19 in. deep by 17¾ in. wide spanning normal to the abutments, with solid concrete infilling and a topping of structural concrete 3½ in. thick. The beams are prestressed with wires of 0·276 in. diameter placed with a gap of 1 in. between them to allow for lateral winding and other irregularities; the wires in these beams were pre-tensioned. A similar gap between the ends of the beams and the walls of the abutment was grouted.

The deck is prestressed transversely with twenty-three high-tensile bars of 1 in. diameter parallel to the abutments, holes and ducts of 2 in. diameter having been provided for this purpose in the webs of the beams and the infilling between the beams. Because of its width (266 ft.) it was impracticable to lay and prestress the entire deck in one operation, and it was therefore divided into five bays of 44 ft. and one bay of about 46 ft.; jacking beams were incorporated between the bays.

The concrete infilling to the deck was placed in two courses. In order to eliminate transverse eccentricity, the first course was centred on the prestressing bars, and the bars were placed and tensioned before the second course of concrete in the beams and the topping were placed. Construction of each bay proceeded continuously in this manner, and the transverse bars between bays were connected by couplers. Provision was made at one end of the deck for future extension to accommodate a second carriageway.

At the ends of the bridge solid parapet walls 6 ft. high are provided to allow for future electrification of the railway. Precast concrete smoke-deflectors are also fitted, and the underside of the deck is painted with bituminous emulsion. All the concrete cast in place contained an air-entraining agent.

The specified crushing strengths of the concrete in the prestressed beams were 6000 lb. per square inch when the tensioning force in the wires was released and 7500 lb. per square inch at 28 days. The maximum working stress is 2500 lb. per square inch and the beams were designed so that there would be no tensile forces.

Work began in October, 1957, and was completed in November, 1958, for the Scottish Home Department (Roads Division). The consulting engineers are Messrs. Babtie, Shaw & Morton and the general contractors Messrs. Farrans, Ltd.

Footpath added to Stone Bridge.

THE footpaths on each side of the bridge over the Thames at Staines are being increased in width from 4 ft. to 8 ft. by supporting precast concrete slabs on cantilevered beams. The new work comprises longitudinal beams, one of reinforced concrete fixed to the stone of the bridge and the other of structural steel at the outer edge to carry the new parapet, and structural steel cantilevered beams between the longitudinal beams. Structural steel beams were preferred to concrete because of the advantages of bolted connections. These beams were encased in concrete.

It was necessary to use concrete of high density in the beam fixed to the bridge, and this was achieved by the use of the noses of 37-mm. shells in concrete made with 3 parts of $\frac{3}{8}$-in. aggregate, 2 parts of sand, and $1\frac{3}{4}$ parts of cement by volume and with a compressive strength of 2800 lb. per square inch at seven days. This concrete had a density of up to 280 lb. per cubic foot, and as the shell-noses, which weighed about 2 lb. each, were placed by hand the density could be varied as required in the length of the bridge. The concrete was compacted by vibration. Experiments made with nut-punchings and cast-iron borings produced concrete with densities up to 230 lb. per cubic foot.

The footpath was formed with hollow precast concrete units. Precast concrete beams, generally 12 in. wide by 18 in. deep and 9 ft. long, were used for the parapet beam, and were reinforced with steel sections bolted to short lengths of structural steel bolted to the longitudinal beam fixed to the bridge.

Fig. 2.

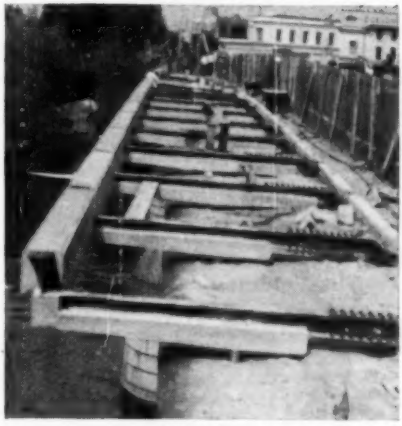


Fig. 3.

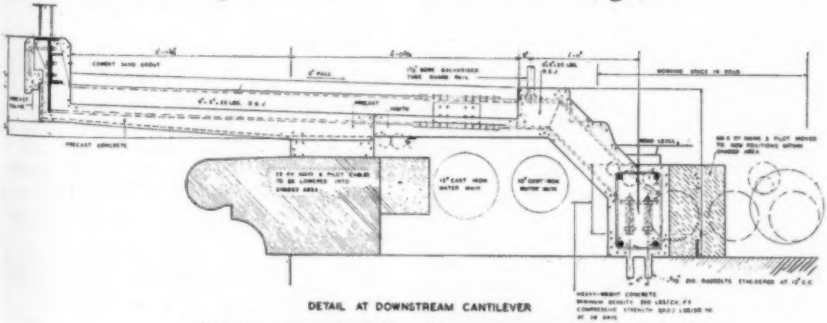


Fig. 1.—Detail of Cantilevered Footpath.

Fig. 4.

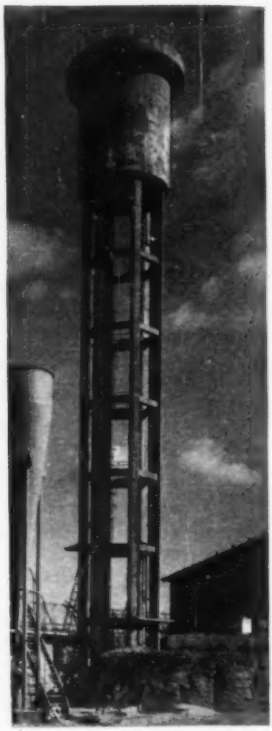
Fig. 1 shows the arrangement, *Fig.* 2 shows the structural steel cantilevers projecting from the reinforced concrete beam fixed to the stone bridge, *Fig.* 3 shows the transverse beams encased in concrete and the new parapet beam, and *Fig.* 4 is a view of the completed work.

Messrs. Leslie Turner & Partners acted as consulting engineers for the Ministry of Transport and Civil Aviation, the contractors were Messrs. A. E. Farr, Ltd., and the precast members were made by Concrete, Ltd.

The Volume of Construction.

THE latest report of the Ministry of Works on the construction industry states that production in the third quarter of the year 1958 was the same as in the third quarter of 1957 and the second quarter of 1958. The value of work done in the third quarter of 1958 was £548,000,000, compared with £537,000,000 in the third quarter of 1957 and £544,000,000 in the second quarter of 1958. There has been a slight reduction of house building during the period, but the value of other new work (apart from repair and maintenance) increased from £211,000,000 in the third quarter of 1957 to £221,000,000 in the second quarter of 1958 and £226,000,000 in the third quarter of 1958, and the number of men employed on such work increased from 442,000 in October 1957 to 448,000 in July 1958 and 349,000 in October 1958. The cost of building rose by less than 1 per cent. in the past year.

Water Tower at Haifa.



THE illustration shows a water tower of 33,000 gallons capacity which was recently erected at Haifa, Israel. The design allows for the future construction of more tanks at intermediate levels, and two-story offices, with access stairs, at the bottom of the tower.

The foundations are on a thick layer of sand about 5 ft. below sea-level. The bottom of the tank is 115 ft. above ground. The design allows for a pressure due to wind of 41 lb. per square foot, a range of temperature of 100 deg. Fahr., and a horizontal acceleration due to earthquake of 0·1*g*. The minimum specified cube strength for the concrete was 6600 lb. per square inch at 28 days.

The tower was designed in one week and built in twelve weeks. The consulting engineer was Mr. I. Olexinzer, of Jerusalem.

New Steel-Strip Mill at Sheffield.

THE strip mill described in the following has been built at Brinsworth, between Sheffield and Rotherham, for the Steel Peech & Tozer branch of United Steel Companies, Ltd. The cost was about £3,500,000 and the mill is designed to produce strip up to 18 in. wide in various qualities, including mild and stainless steels. A pickling plant is included and provision has been made for cold rolling the product of the works at a future date.

The civil engineering work commenced in July 1955 and the first trial run of the mill took place on June 10, 1957. Miscellaneous extra work and the pickling plant were completed in January 1958, when the mill was in regular production.

The site permitted access from the works railway already existing to the east, but considerable excavation and the removal of most of one side of a hill was necessary. *Fig.* 1 shows the site shortly after excavation commenced; the lowest part is 70 ft. below the top of the hill in the background. The subsoil consists largely of strata of carboniferous sandstone, shale, and siltstone. Some of the siltstone, although of adequate bearing capacity in the natural state, deteriorates on exposure. At the low (north-west) part of the site an alluvial deposit of poor bearing value overlies the carboniferous strata. To avoid risk of settlement nearly all the foundations were taken down to rock.

The first contract was for a road to give access along the north of the site. The second was for the excavation, including the railway cutting and the formation for the railway tracks. The excavation was done in two stages to allow the foundations of the mill to be started promptly, and also to permit the diversion of the main water supply, which crossed the site, to a new position; this diversion was the subject of a separate contract.

Excavation and Levelling.

Large quantities of excavation had to be removed from the site, but a small part was used to raise levels. Much of the excavation was in rock, which on part of the site was 15 ft. below the original ground level. Under the slab bay (*Fig.* 3), because of the high loading to be expected (often a stack of steel slabs 12 ft. high), trenches 16 in. wide were excavated through the alluvial deposit to rock at 7 ft. centres and filled with plain concrete. Similar trenches were provided under the furnaces and other heavy plant as shown in *Fig.* 2. For smaller loadings bored piles were used. Over nearly half the site of the main building no excavation was necessary, but more than 450,000 cu. yd. of excavation was required over the southern part of the site and in the railway approach, which is in a cutting for almost its whole length. Of this quantity about 325,000 cu. yd. was rock, in which blasting charges were inserted in holes drilled from above and near to the working face with microsecond delays between firing adjacent holes in order to avoid risk of damage by earth shock to structures some distance away. The charges varied, depending on the hardness of the rock, from 30 lb. upwards of open-cast gelignite per hole; not more than 150 lb. were detonated at a time.

Before excavation started, land drains were laid generally across the site from east to west and the water was collected by an open main drain to the west of the site.

Concrete Work.

The substructures of the slab-reheating furnaces, flues, recuperators, chimney, roughing-mill stands, run-out table, and the complicated basement construction shown in *Fig.* 2, together with the forced-air duct, scale-water flume, transformer pits, and the main building foundations, were carried out in one contract. Foundations for the finishing stands, coilers, and the conveyors taking the coils to the coil-processing bay, were placed with another contractor in order to expedite the work.

The foundations for the mill stands are generally large blocks of concrete lightly reinforced and penetrated by ducts, trenches, and tunnels for services, cooling-water flumes, and access to the bearing plates for the holding-down bolts.

The concrete was generally a 1 : 2 : 4 mixture, and weigh-batching plants were

Fig. 1.—Levelling the Site.

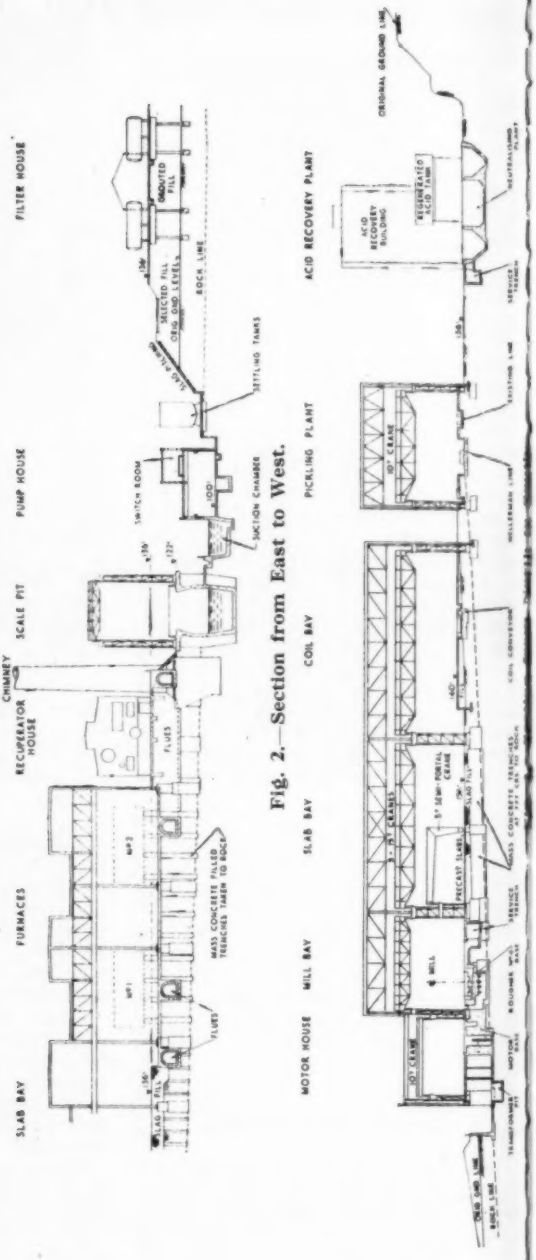


Fig. 2.—Section from East to West.

used; concrete pumps were used on part of the work. The coarse aggregate was 1 in. maximum size for plain concrete and 1 in. and ¾ in. for the reinforced concrete. The total quantity of concrete placed exceeded 30,000 cu. yd.

The external faces of basement walls were treated with bitumen to prevent the penetration of dampness; no water pressure was expected because of the land drains provided, but all structures below ground were designed to allow for possible flooding of the sub-soil. The deeper pits for the transformers are lined with asphalt. For the bedding of the bases of roll stands and other plant expansive cement grout was used to avoid uneven support by shrinkage; for concreting holding-down bolts in their ducts or pockets, however, fine concrete or ordinary cement grout was used.

The foundations for the roll grinders are separated from the remainder of the structure by casting them on studded rubber matting with a 3-in. air gap at the sides to isolate the machines from the mill vibrations so that the rolls could be accurately ground.

The scale pit measures 88 ft. by 20 ft. by 34 ft. deep and the pumphouse measures 80 ft. by 36 ft. The bottom of the pit is 45 ft. and the floor of the pumphouse is 36 ft. below the general floor level of the mill. The filter house is on 12 ft. to 15 ft. of hard filling, the filter tanks being supported by columns built before the filling was placed; the filling was consolidated with injected grout to reduce possible settlement.

The brick chimney for the slab-reheating furnaces is 160 ft. high and is adjacent to the excavation for the scale pit; 508 cu. yd. of plain concrete were used to extend the base of the chimney to rock.

A concrete service trench, generally about 6 ft. wide by 8 ft. deep, was required for pipework and cables, extending the full length of the mill. Similar service trenches extend to the pump house, filter house, and cooling-water tower, and also across the mill to the pickling plant. A service trench also extends to the north-west corner of the mill under the Sheffield road and another road from the main works. It was not permitted to disturb the Sheffield road, so a tunnel was formed under it consisting of precast concrete sections 8 ft. in diameter bolted together; the outside was grouted to prevent settlement of the road and leakage of sub-soil water into the tunnel. The other tunnel was rectangular; it was cast in place in excavation.

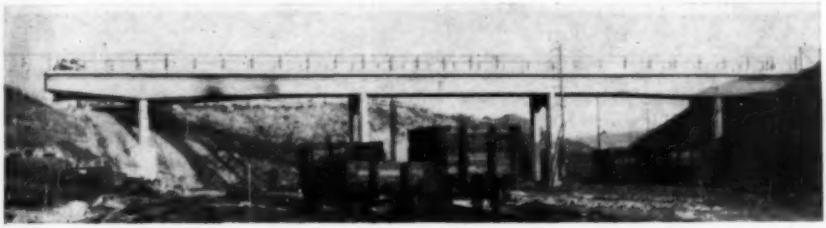


Fig. 4.—Prestressed Bridge.

The floor of the motor room is covered with oil-resistant tiles. The floor adjacent to the mill stands has a non-slip topping and was treated with a chemical hardener.

In the slab-storage bay, where the load may be up to 1½ tons per square foot, 9 in. of ungraded slag was covered with 6 in. of crushed slag, and blinded with dust, except for the area where the slabs are prepared which was covered with precast concrete slabs 4½ in. thick to allow for any subsequent relevelling that might be necessary.

In the coil bay the part of the floor which had been excavated was finished with heat-resisting tiles laid on a concrete slab. The part which had to be filled to level was finished with concrete slabs laid on sand and slag so that a level surface could be restored in case of settlement.

The pickling plant and the acid-recovery treatment plant are to the south of the mill. One row of pickling tanks had to be moved from the existing works with as little interruption as possible and

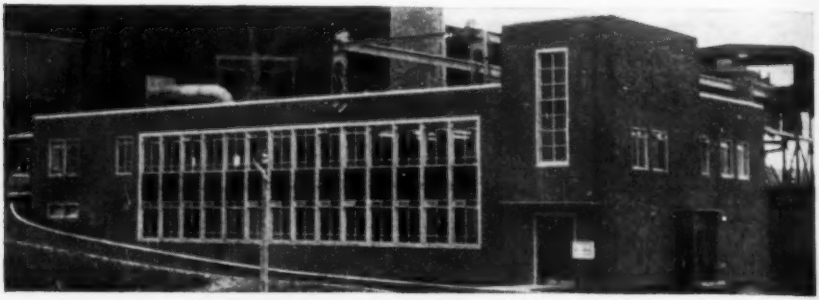


Fig. 5.—Office and Amenity Building.

the work for the second row of tanks completed while the first row was in operation. In case acid should come into contact with the foundations, sulphate-resistant cement was used.

The bridge shown in *Fig.* 4 was built before the excavation was carried out below for the railway cutting, and was described in this journal for January 1957.

The office and amenity building (*Fig.* 5) provides offices on the first floor and amenities for the operatives on the lower level.

The building is carried on 65 17-in. diameter bored concrete piles taken to rock about 18 ft. down. After extending and placing filling around the piles to raise levels, the ground-floor slab and beams were constructed incorporating a duct for heating and other pipes for the whole length of the building. The first floor and roof are of solid reinforced concrete slabs and beams carried on reinforced concrete columns. The concrete for the piles was a 1 : 1½ : 3 mixture and all other reinforced concrete work was a 1 : 2 : 4 mixture. A plasticiser was added to the mixture for the superstructure to assist placing the stiff concrete, which was compacted by internal vibration. The window surrounds and mullions are of cast stone. As a protection against damp, and to retard discoloration by smoke and dust, all external brickwork and cast stone were sprayed with silicone.

The total cost of the civil engineering work and the amenity and office building was slightly over £1,000,000.

The work was carried out under the direction of Mr. V. Watkins (Chief Engineer); Mr. J. Waller and Mr. L. Parkin were responsible respectively for the mechanical and electrical work. Mr. Donovan H. Lee, M.I.C.E., was the consulting engineer for the design and supervision of the civil engineering work. Sir Alfred McAlpine & Son, Ltd., were the contractors for the excavation and levelling, and for part of the substructure and the scale-water pit. The main concrete work for the mill building was done by Messrs. George Longden & Sons, Ltd., and the substructure of the pickling plant and the tunnels were built by Messrs. Henry Boot & Sons, Ltd. Messrs. Bovis, Ltd., built the amenity and office building. The bored piling was by the Cementation Co., Ltd., who were also responsible for consolidating the filling under the filter house. The superstructure is of structural steel, designed and supplied by United Steel Structural Co., Ltd.

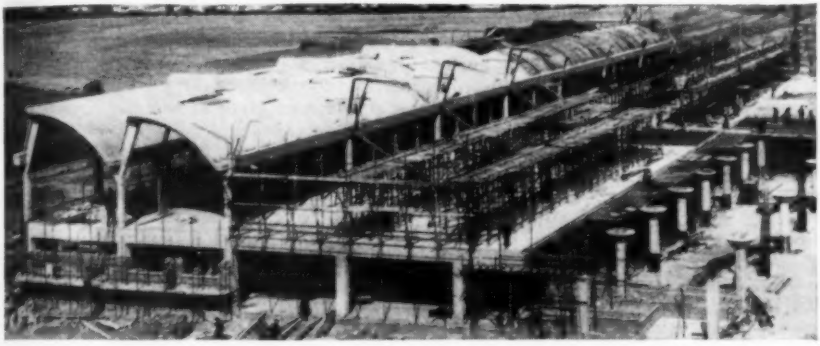
A New Shuttering System.

A SYSTEM of shuttering known as "V-form", which has been used in Canada and the U.S.A., is now available in this country. The shuttering comprises plywood panels, which are protected around the edges by mild steel angles and welded to frames of mild steel channel. The shuttering is supported on steel joists, on which are steel pins which engage holes in the frames of the shutter panels and in the head-plates of adjustable shores which support the beams. The system was developed by Acrow, Ltd., from whom full details can be obtained.

Large Factory with North-light Roofs.

A FACTORY for Messrs. Cadbury Bros., Ltd., has been under construction at Moreton, Wirral, Cheshire, for the last few years and now comprises some 500,000 sq. ft. of buildings.

The illustrations are of a two-story building with an area of 70,000 sq. ft. The first floor is of flat-slab construction 12 in. thick supported on mushroom-headed circular columns. The columns were cast in steel shutters made in halves and bolted together. The shuttering for the floor was supported on mobile scaffolding which was lowered on to wheels, pushed over the floor, and jacked up to a new position.



The roof is of north-light " shell " construction in bays with a chord width of 25 ft. and 175 ft. long with a central expansion joint. The shells are supported at 50 ft. centres, alternate valley beams being supported from a reinforced concrete girder of 50 ft. span. Plywood centering was used, supported on timber trusses and scaffolding.

The architect is Mr. C. J. Wilkinson, chief architect of Messrs. Cadbury Bros., Ltd. The concrete design is by G.K.N. Reinforcements, Ltd. The general contractors are Messrs. A. Monk & Co., Ltd.

Precast Frame Construction.

A BUILDING now being erected for the London Electricity Board in Cambridge Heath Road, London, E., is 160 ft. by 40 ft. by five stories high with a single-story annexe 138 ft. by 34 ft. in plan.

The frame is of precast reinforced concrete columns and main beams with small cantilever beams at each end of the main beams to support balconies on each side. The frames (*Fig.* 1) formed by these columns and beams are tied longitudinally by edge-beams and prestressed precast floor joists; concrete blocks are laid between the joists.

A tower crane was used for erecting the members, which range in weight from 4 tons to 7 cwt., and for hoisting the concrete cast in place in the end towers and balcony walls.

The architect is Mr. Leslie K. Watson of Messrs. Watson & Coates, and the consulting engineer is Mr. Cyril Blumfield. The whole of the reinforced concrete work above ground level, including the casting of the frames, is being constructed by Messrs. F. Bradford & Co., Ltd. The main contractors are Messrs. Tersons, Ltd.

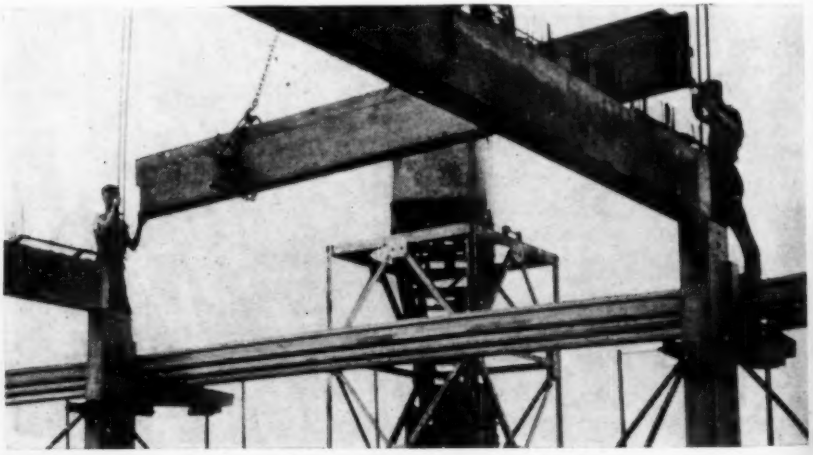


Fig. 1.

Fire-Testing Station, Elstree.

NEW "BURNING" BUILDING.

THE building shown in *Fig.* 2 has been completed recently at the Fire-Testing Station at Elstree, Herts, for the Department of Scientific and Industrial Research. It is designed for testing the fire resistance of full-size and nearly full-size structural members and building materials of sizes that it has not before been possible to test in this country. The building is also arranged for other types of testing and is provided with quenching facilities. The main building is 135 ft. by 52 ft. and has a clear height of 38 ft. ; additional offices, a pump-room and a tank are provided at one side. The central part of the roof is normally slid open when fire tests are being made.

The soil is brown clay of medium strength, and spread foundations are used to provide a maximum bearing pressure of 2½ tons per square foot when the building is subjected to the maximum effects of wind and temperature. The settlement of the foundations was measured by the Building Research Station, and was found to be 0·3 in. initially and 0·7 in.

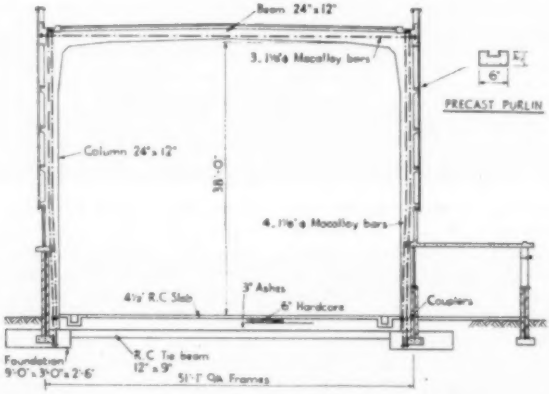


Fig. 1.—Details of Frames.

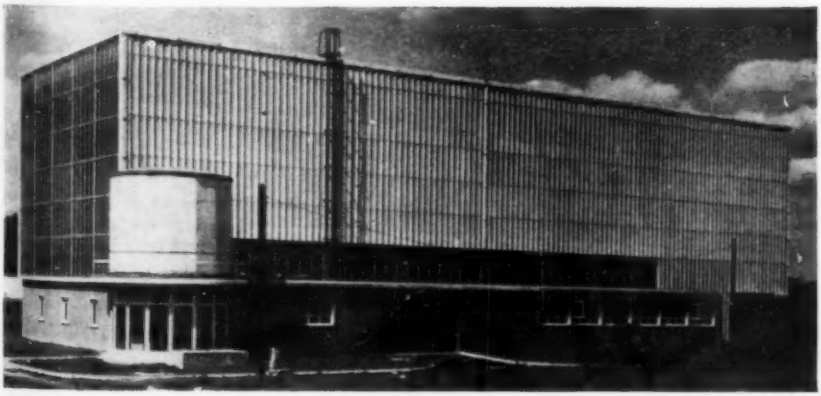


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Fig. 2.

finally for a mean pressure, without wind, of about 1 ton per square foot.

The main framework consists of cast-in-place frames fixed at ground level and pre-stressed with Macalloy bars. The roof is laid to falls, and comprises Stahlton hollow-tile slabs covered with insulating concrete 3 in. thick and 3-ply bituminous felt. The effect on the frames of a rise in temperature of 200 deg. C. was found to be about the same as that of wind pressure. A comparison of costs favoured cast-in-place construction compared with an alternative design of precast units pre-stressed together at the site. Both ends of the building are fully glazed in frames of light-gauge steel, using tubular members to a large extent.

At the eaves and at a lower level reinforced concrete tie-beams are provided to the frames. The prestressed purlins (*Fig.* 1) were made by the pre-tensioning process. Diagonal bracing of steel angles is provided in two bays near the ends of the building, and there is an expansion joint across the centre. Thermocouples are attached to some of the prestressing bars, and at a later date the Building Research Board may publish observations on the strain-gauge readings.

The concrete for the frames was specified to have a maximum water-cement ratio 0·42, a maximum slump of ¾ in., and a minimum compressive strength of 6500 lb. per sq. in. at 28 days. The total cost of the building was about £44,000.

The building was designed for the D.S.I.R. by the Directorate of Works of the Ministry of Works. Mr. Donovan H. Lee, M.I.C.E., was consultant for the structural design in collaboration with the Chief Structural Engineer's Branch of the Ministry. Messrs. Tersons, Ltd., were the contractors.

The Effect of Entrained Air on the Strength of Concrete.

An investigation has been made by Mr. B. W. Shacklock and Mr. P. W. Keene (of the Cement and Concrete Association) on the compressive and bending strengths, after curing in water or air, of concretes of similar cement content and workability with and without entrained air. Other properties measured were bulk density, ultrasonic pulse velocity, dynamic modulus of elasticity, and change of weight during curing. Among the conclusions reached were the following.

The air-entrained specimens had less tendency than the plain concrete specimens to increase in weight during water curing, to decrease in weight during air curing, or to decrease in compressive and bending strength when cured in air instead of in water. From this it appears likely that air-entrained concrete would be less susceptible than plain concrete to unfavourable curing conditions.

The compressive and bending strengths of the air-entrained concrete were generally less than those of plain concrete of similar cement content and workability, but the differences in the bending strengths of air-dry specimens were negligible. The compressive and bending strengths were 8 per cent. and 30 per cent. lower in the case of specimens cured in the air of the laboratory and tested dry than in the case of specimens cured in water and tested wet.

There were no significant differences in bending strength between beams made in a laboratory and tested according to Part 11 of British Standard No. 1881: 1952 and others tested on their sides; the tests did not include beams made under site conditions, in which case a greater variation of results might result in the case of beams tested on their sides.

Design Tables for Prestressed Beams.

A book of design tables for use with the standard prestressed beams which they supply has been issued by Leap Concrete, Inc., of Lakeland, Florida, U.S.A. (price 2 dollars). The beams are of double-tee cross section, with a top 5 ft. wide and two stems 18 in. deep with three pre-stressing strands in each.

Bridge at Bridgwater, Somerset.

THE bridge (*Figs.* 1 and 2) over the River Parret at Bridgwater was designed for Ministry of Transport Heavy Loading. It has a total length of 160 ft., consisting of a 70-ft. central span and two 45-ft. end spans, and is 60 ft. wide between the parapets.

The site comprises a soft alluvial deposit over about 20 ft. of softened Keuper marl, and cast-in-place piles of 18 in. diameter were used with a working load of 50 tons. Vertical and raking piles support the abutments, which are of the counterfort retaining-wall type. The piers are of reinforced concrete, the horizontal thrust on which is negligible. Owing to the 15 deg. skew of the bridge, continuous concrete bearings would have been subjected to considerable torsion; mild steel bearings surrounded by steel boxes packed with grease were therefore used, with their axes normal to the longitudinal axis of the

Fig. 1.—Blake Bridge, Bridgwater.

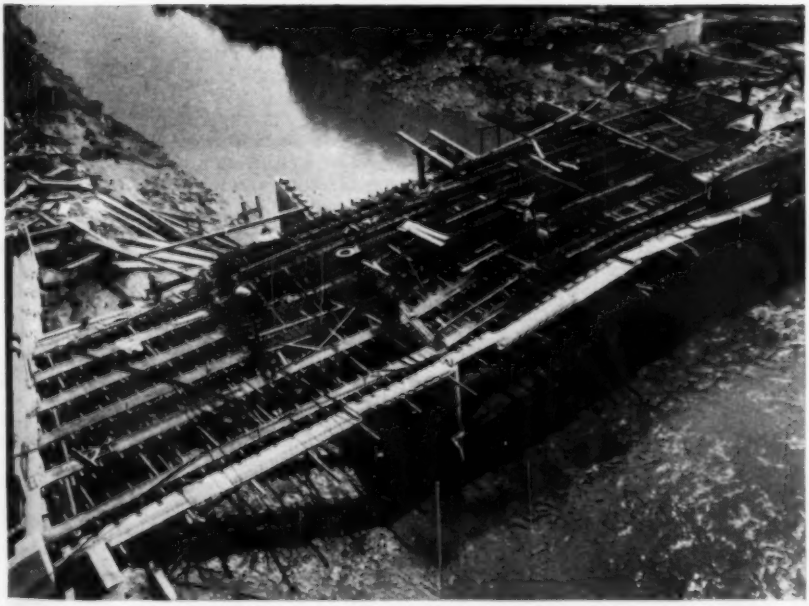


Fig. 2.—View during Construction.

bridge. The rollers consist of bright steel bars of 6 in. diameter. In order that dredging may be done in future, the abutments and the piers were taken to a lower level than is necessary at present.

The deck slab is 9 in. thick and is supported by eleven beams 24 in. wide, spaced at 5 ft. centres; in the central span the beams are 34 in. deep at the centre and 70 in. deep over the piers. There are transverse beams between these beams. It was not possible to avoid a combination of skew and fall in the bridge, which is on a straight longitudinal axis; the curve of the road is formed by a slight variation in the footpaths.

The work was carried out for the Somerset County Council (Mr. J. H. H. Wilkes, County Surveyor) by Sir Lindsay Parkinson & Co., Ltd., and Messrs. L. G. Mouchel & Partners were the consulting engineers.

Bridge with Heavy Prestressed Precast Beams.

THE illustration shows the central span of a prestressed bridge over the railway at Tuffley, near Gloucester, and which forms part of a new by-pass road being built under the supervision of the City Engineer of Gloucester.

The bridge has a cantilevered span at each end which carries a simply-supported main span. The end spans cantilever 14 ft. from the piers and the main span is 66 ft. long, so that the clear distance between the piers is 94 ft. All the beams were precast and were cast behind the abutments. The cantilevered beams, which weigh 27 tons, were cast in shutters over a narrow-gauge railway track. After they were prestressed the beams were jacked up, lowered on to bogies, and pulled forward with the aid of a winch over the track, which was extended and supported on scaffolding over the gap between the abutment and the pier. When the beams were in position they were lowered on to the supports.

For the erection of the suspended central beams, which weigh 30 tons each, a Bailey bridge was erected across the gap, and two gantry cranes, travelling on rails on top of the Bailey bridge and overhanging its sides, lifted the beams (which had previously been pulled forward in the same way as the cantilevered beams) and lowered them into position. Transverse beams between the longitudinal beams, and also the deck slab, were cast in place and the deck prestressed transversely with the aid of a cradle overhanging the railway to accommodate the men and equipment.

All the work over the railway was done on Sunday mornings with occupations varying between 1½ hours and 3 hours. The Freyssinet system of prestressing was used throughout.

The bridge was designed by Messrs. Scott & Wilson, Kirkpatrick & Partners in collaboration with Messrs. Ansell & Bailey, consulting architects. The general contractors were Messrs. A. Monk & Co., Ltd.

Precast Concrete in an Office Building.

An office building (*Figs.* 1 and 2) now being erected at Stanlow oil refinery, near Chester, for the Shell Petroleum Co., Ltd., will be 100 ft. high, 285 ft. long, and 40 ft. wide. It will have eight stories and a basement, and reinforced and prestressed concrete are used throughout. The building will provide accommodation for 400 people, and can be extended, if required, to increase the volume by 25 per cent. Two access areas are provided, each of which includes a staircase and lift. All the vertical service ducts are also installed in these areas.

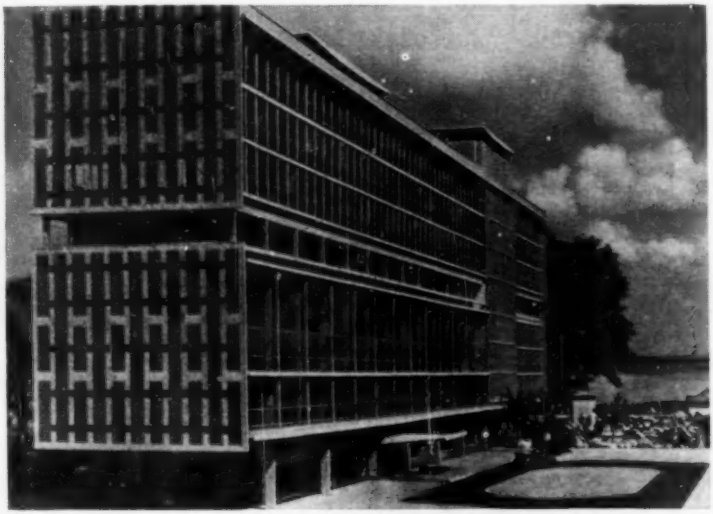


Fig. 1.—Front Elevation.

Each office is 16 ft. long and a multiple of 4 ft. 9 in. in width. Permanent partitions of lightweight concrete blocks are used, on the ground that it is easier and cheaper for the staff to move to different rooms than to change the positions of partitions; also permanent partitions may be more resistant to sound than removable partitions.

The loading due to wind is resisted by the walls of the access areas, to which the forces are transmitted by the floors acting as girders. The concrete for these towers, and for the basement and supporting framework up to first-floor level, is cast in place; all other main structural members are precast. The building is supported on piles, each of which can carry a load of 60 tons.

The floors are 8 in. deep and consist of inverted tee-beams of prestressed concrete at centres of 2 ft. 4½ in. with a topping at least 2 in. thick cast in place on permanent shuttering of corrugated asbestos-cement sheets supported on continuous rebates in the webs of the beams. Castellations on the top of the beams provide a bond between the precast and in-situ concrete. The floor slabs span from edge-beams on the longitudinal walls to an internal beam; the spans of the floors are 23 ft. 9 in. and 16 ft. 5 in. There are no downwardly-projecting transverse beams, so that the partitions can be where required and the central longitudinal beam forms part of the wall of the corridor. The construction of the roof is similar, except that deeper prestressed tee-beams at centres of 4 ft. 9 in. span the full width of the building.

Precast concrete frames one story high, each comprising two columns at centres of

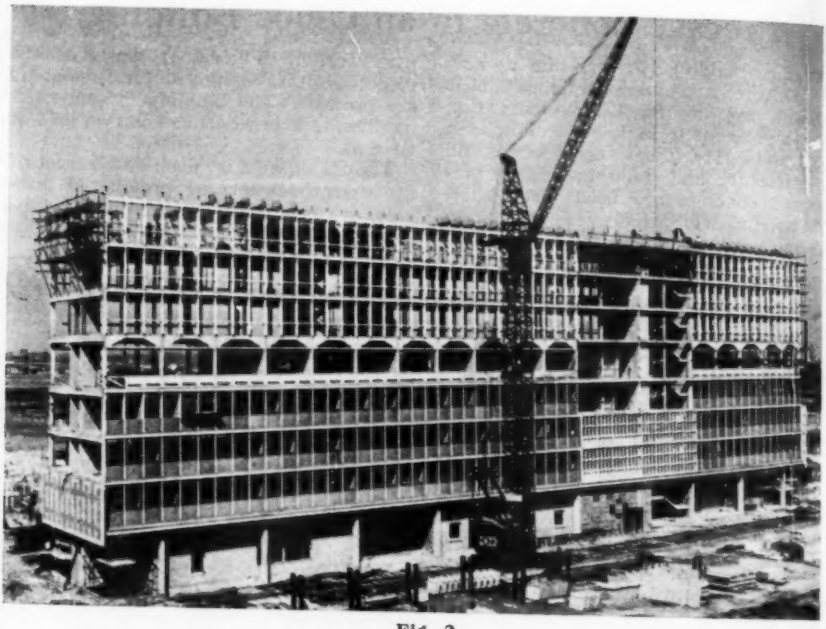


Fig. 2.

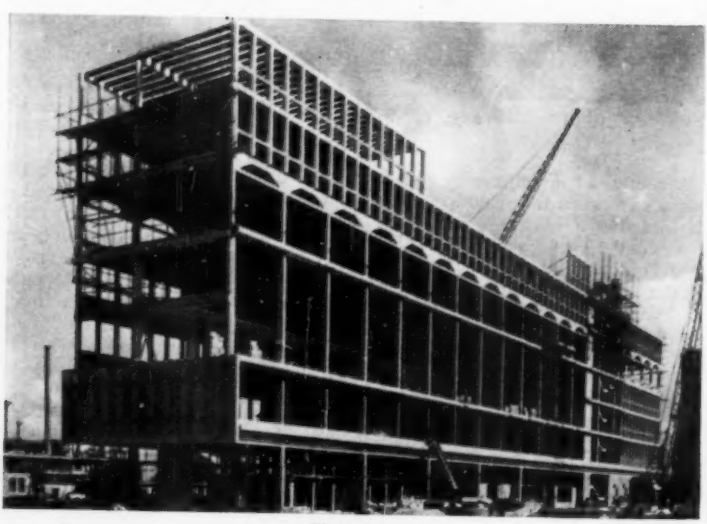


Fig. 3.

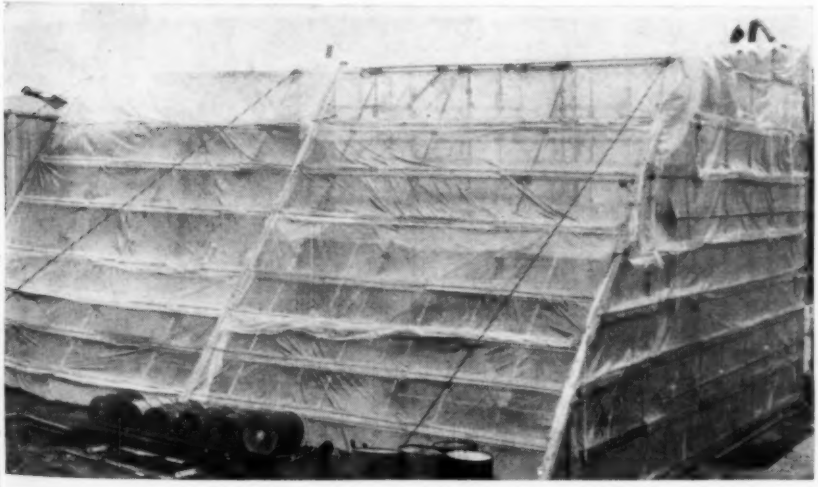
4 ft. 9 in., a head-beam, and a sill-beam, form the external walls (*Fig.* 2). The frames are joined by precast infilling head and sill beams, which are bolted to them. The columns are bolted together within the depth of the floor slab, and a narrow strip of the floor topping is left until the frames for the next story have been erected. When the frames are levelled and all bolted connections tightened the joints at the bottoms of the columns and the remaining topping are cast. Longer precast columns are used at centres of 14 ft. 3 in. for a part of the second and third floors (*Fig.* 3). The internal supporting structure consists of precast columns and beams, except in the access areas, which are cast in place.

A white stone finish is provided on the columns of the single-story frames, and the infilling panels are faced with grey-green exposed granite aggregate. Bands of concrete faced with exposed Derbyshire spar are provided at the level of each floor. The access areas are faced with grilles formed of precast units. At the ends of the building are concrete panels faced with exposed brown gravel and grey Cornish granite and quartz.

The cost of the work is expected to be about £700,000. The architects are Messrs. Grenfell Baines & Hargreaves, the consulting engineers are Messrs. F. J. Samuely & Partners, and the general contractors are Messrs. A. Monk & Co., Ltd. The prestressed floor and roof units are supplied by Pierhead, Ltd., the precast wall frames by Messrs. George Greenwood & Sons, and the facing slabs by Forticrete, Ltd.

Use of Polythene for Protection against the Weather.

THE illustration shows the use of polythene sheets to form a draught-proof screen during the welding of an aluminium structure. The volume of the shelter was 110,000 cu. ft., and during the two months when it was in use the polythene withstood the effects of gales and thunderstorms. The manufacturers (British Visqueen, Ltd., of Stevenage) state that the material can also be used as an underlay, a separating medium, or a cover for curing concrete slabs.



Land-drainage Pumping Station near Boston.

THE Hobhole pumping station and sluice (*Figs.* 1 and 2) recently constructed near Boston, Lincs, for the Witham Fourth District Internal Drainage Board is designed to discharge 1000 cu. ft. of water per second at 12-ft. static head from the Hobhole drain into the tidal river Witham.

The structure is founded on 394 piles 14 in. square, including 117 rakers with an average length of 27 ft. driven into the underlying stiff blue clay. The base-slab supported by the piles is 100 ft. by 120 ft. by 4 ft. thick, and is entirely surrounded by steel sheet piles also driven into the clay. On this slab are reinforced concrete piers forming the water channels and supporting the large block of concrete, up to 15 ft. thick, containing three 87-in. pumps and their volutes which measure about 18 ft. across at the widest point. The top of the slab forms the floor of the engine room; as this floor is 13 ft. below the highest known tidal level, the south

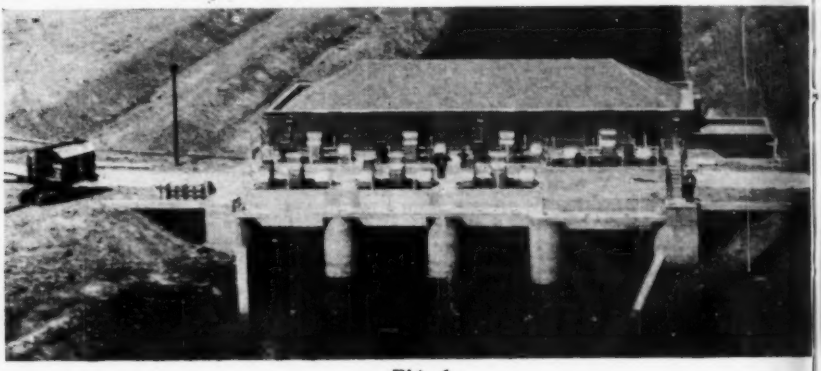
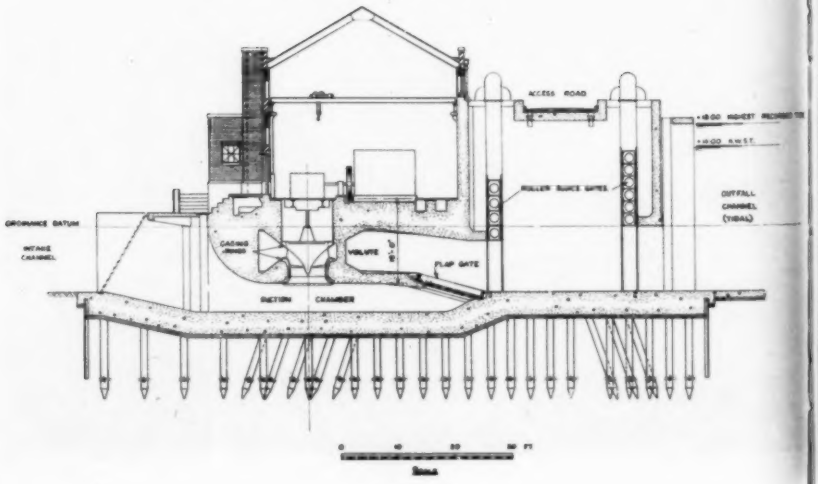


Fig. 1.



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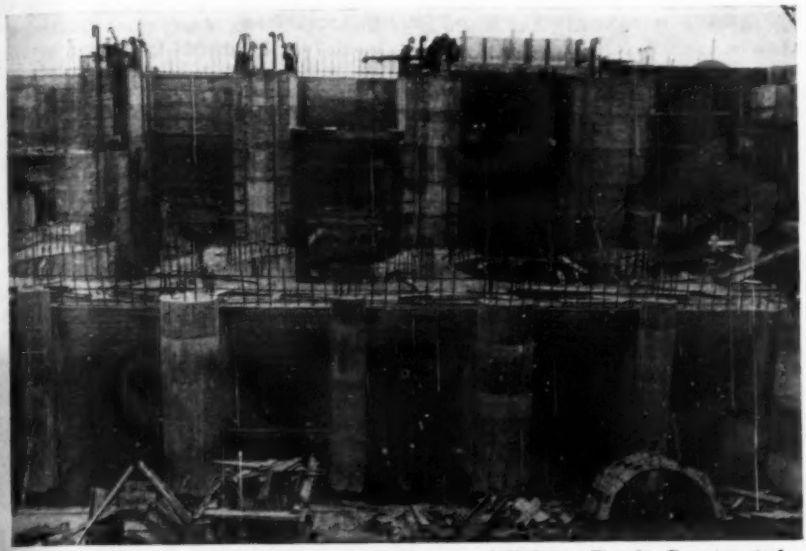

Fig. 3.—Intake to Suction Chamber: Piers and Volutes Partly Constructed.

Fig. 4.—Interior of Volute, showing Upper and Lower Cones and Casing-ring.

wall is designed as a bulkhead wall spanning between the piers of the discharge channels.

The piers forming the discharge channels (*Fig.* 3) extend about 3 ft. above tidal level and support an access road 12 ft. wide. At the outer end of these piers is a second bulkhead wall which, combined with vertical roller sluices in each channel, forms a second line of defence against tidal flooding. The structure incorporates a channel 17 ft. wide by 16 ft. 6 in. high for discharging water by gravity at low tide; this channel is closed by a counterbalance cycloidal gate and a vertical roller-sluice. An unusual feature is the sealing of the rear of the suction chamber by an automatic flap which, when open, allows additional discharge by gravity.

In spite of their complicated shape, the pump volutes (*Fig.* 4) were formed in concrete with a variation of no more than ¼ in. from the designed shape. The pump casing-rings were cast into the concrete as the work proceeded, and upon the accuracy of the placing of these parts depended the alignment of the pumping machinery. The lower cone of the volute presented some difficulty in concreting as the top of the cone is surmounted by the lower cast-iron casing-ring; at this point special steel shutter panels were used and secured to the bottom of the volute and, by special clips, to the casing-ring, the space to be concreted being packed with 3-in. to 4-in. gravel which was grouted with colloidal cement-mortar through holes in the casing-ring.

The walls of the volute were next cast in one lift of 8 ft., followed by the roof and the top cone. The upper casing-ring, which is cast into the top cone and carries the whole weight of the rotating parts of the pump, was secured during this operation by a special jig supplied by the makers of the pump.

The superstructure is a reinforced concrete frame with 9-in. brick panels covered with 4½-in. facing bricks.

The engineer of the Board is Mr. G. E. Buchner, M.B.E., A.M.I.C.E. The consulting engineers for the foundations and reinforced concrete work were Messrs. L. G. Mouchel & Partners. The contractors were Messrs. A. E. Farr, Ltd. The pumps were made by Messrs. W. H. Allen, Sons & Co., Ltd., and the sluice gates by Messrs. Ransomes & Rapier, Ltd.

FIFTY YEARS AGO.

From "Concrete and Constructional Engineering", January–February, 1909.*

Compaction by Vibration.—An improved form of shaking table has recently been introduced by the Improved Construction Co., Ltd., Westminster, which has many advantages. This oscillates in all directions at once, and does not cause the aggregate to move sideways but only vertically, thus wedging the particles together so as to form a very compact concrete that can be removed from the moulds within about an hour. This gives the advantages of wet moulding while enabling the blocks to be removed from the moulds very soon after they are made, so saving the expense of a great quantity of moulds.

A similar principle of shaking the concrete was applied some time ago by Mr. Richard Bowen, of the Monocrete Construction Company, of Cheltenham, in a machine for moulding concrete blocks, with this difference, that the concrete was not only shaken but was at the same time pressed in the mould. The manner in which the blocks were moulded was for the plunger to work within the mould box, the pressure being applied gradually, while the mould box was sharply shaken by blows being struck against it as the plunger descended. Blocks of very great density and strength were thus obtained.

* "Concrete and Constructional Engineering" appeared in alternate months until September, 1909.

A School near Cardiff.

PROVISION FOR SUBSIDENCE.

THE Ystrad Mynach College of Further Education is being built at Ystrad Mynach for the Glamorgan County Council. Although at present there are no mine workings under the site, precautions have been taken against possible future subsidence by providing a rigid structure bearing on the ground at points where facilities can be provided for raising the structure by jacking should this be necessary.

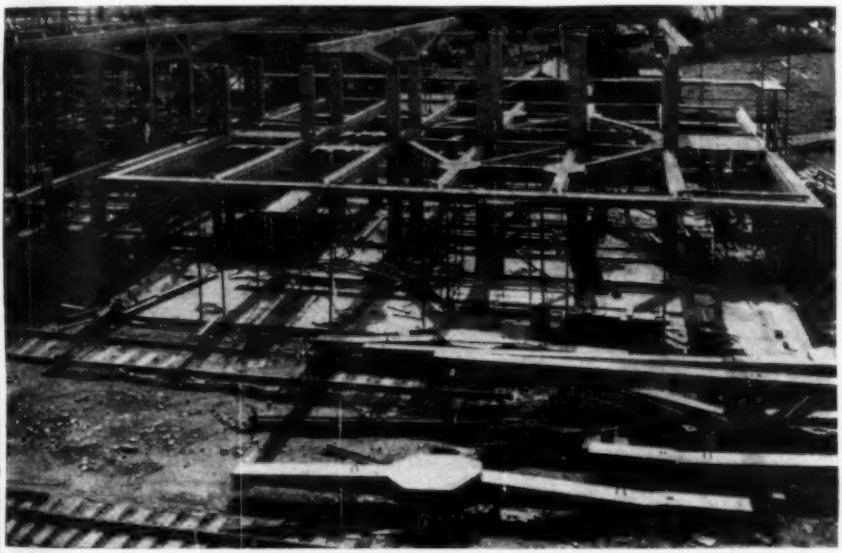

Fig. 1.—Construction of Second Floor, showing Beams around Stair-well at First-floor level. Prestressed Half-Trusses in Foreground.

The teaching and administration building is 136 ft. by 60 ft. 8 in. and four stories high, and consists of two buildings which are entirely separated by a gap of 3 in. The classroom buildings, which are 60 ft. long, are to the west of the joint and the administration building, which is 76 ft. long, to the east. The lower floors provide administrative and communal accommodation, on the third floor are classrooms, and the fourth floor comprises the lecture hall and a tank-room. The rooms are on both sides of a central corridor, and all the floors, including the ground floor, are carried on two rows of reinforced concrete columns cast in place, one row on each side of the corridor. These columns are supported on prestressed foundation beams of hollow cross section which rest on two plain concrete piers. Two points of support are provided on each pier, and provision is made for jacking up the building should settlement of the piers take place.

Each pair of columns carries at each floor level a precast concrete truss 60 ft. 8 in. long cantilevered beyond the column. The depth of the truss (*Fig.* 2) at the columns is 4 ft. 1 in., tapering to 7½ in. at the ends. The top members of the truss were cast and prestressed with Macalloy bars before the remainder of the truss was cast.

In both parts of the building there is a staircase between the central columns; the stairs consist of a central reinforced concrete beam 9 in. wide and between 12 in. and 16 in. deep cast in place, with precast treads and landings supported by

the spine beam and with a concrete connection cast in place. In order to provide space for the stairs the central part of some of the trusses has been omitted. Where this occurs the truss comprises half-trusses, and a prestressed and reinforced concrete lattice construction is cast in place between the half-trusses and the full truss (*Fig.* 1). This lattice construction consists of prestressed diagonal members in the plane of the lower (inclined) members of the trusses. At the lower end of the inclined member a prestressed beam cast in place extends along the sides of the columns. The construction throughout is as light as possible in order to reduce the loads on the cantilevers; most of the floors and roofs are therefore of timber.

All the services are accommodated in

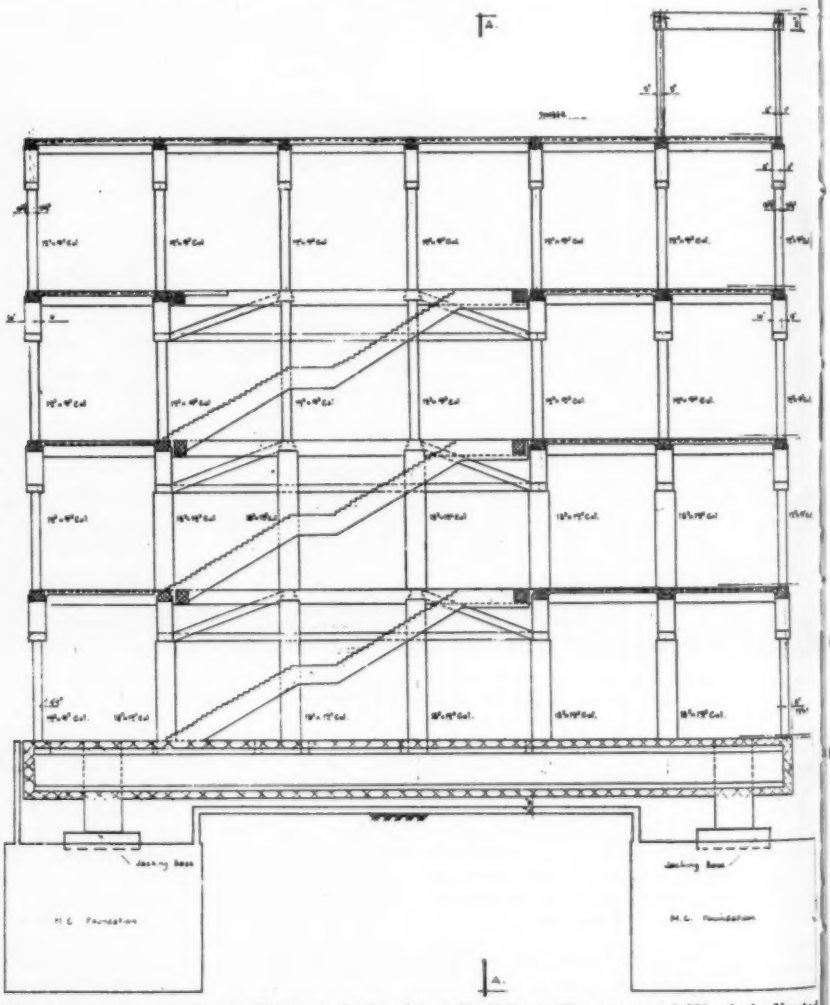

Fig. 2.—Section through Centre of Teaching Building: Prestressed Work indicated by Hatching. *See Fig. 3 for Section A—A.*

the hollow ground-beams (*Fig.* 3) and the floor trusses to permit reduced ceiling heights in the corridors. The trusses for the top floor have a concave camber to allow rainwater to flow to vertical ducts near the centre of the building.

The walls are mainly of aluminium; thermal insulation is provided internally. The cladding to the lowest story is suspended from the floor above, with provision for differential movement. With the exception of the stair treads and landings, all the precast members were made on the site.

The workshop building is 103 ft. long by 60 ft. 8 in. wide. The superstructure is similar in construction to that of the classroom and administration building, except that precast troughs are used for the first floor. In this case the spine columns are supported on concrete foundation trusses. The chord member is of

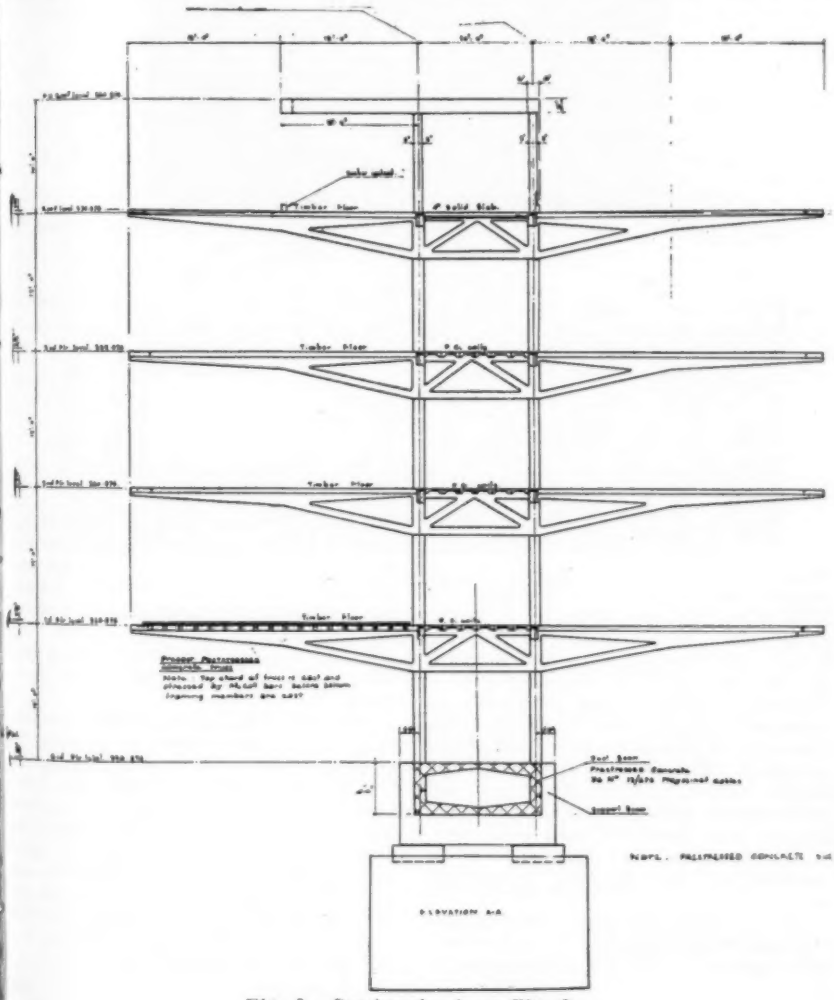

Fig. 3.—Section A—A on Fig. 2.

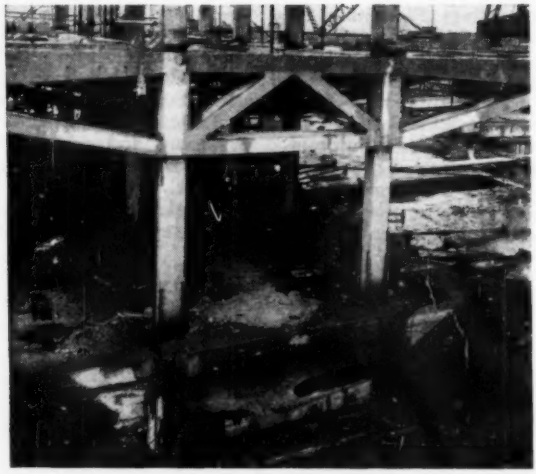

Fig. 4.—End of Spine Construction in Teaching Building : Prestressed Hollow Beam Supporting Columns cast in Place.

hollow cross section and is prestressed by the Freyssinet system. Provision is also made for jacking should settlement occur.

The architects are Messrs. T. Alwyn Lloyd & Gordon, the consulting engineers are Messrs. Felix J. Samuely & Partners, and the contractors are Messrs. William Moss & Sons, Ltd.

Resurfacing Concrete Slabs.

A PAPER entitled "Bonding Thin Concrete to Old Pavements", by W. G. Westall, in the U.S.A. journal "Civil Engineering Magazine" describes the application of new surfaces 1 in. to 1½ in. thick to old concrete at several U.S.A. Air Force bases. The surfaces had become coated with oil and grease. Unsound surfaces were removed by scarifying. Areas not scarified were cleaned with a water-soluble commercial detergent which was scrubbed into the surface with a mechanical sweeper and then flushed with water until a neutral reaction was indicated with pH paper. All the surfaces were then cleaned by sweeping and blowing with compressed air.

The surfaces were then sprayed with commercial muriatic acid (27·9 per cent. HCl) applied at the rate of 1 gallon to 100 sq. ft. Immediately after foaming stopped, the surface was scrubbed and flushed until a neutral pH reaction was obtained. Just before the new concrete was placed, a 1 : 1 sand-cement grout of the consistency of thick paint was scrubbed into the dampened surface with stiff brooms, leaving a coating about $\frac{1}{16}$ in. thick.

The new topping comprised crushed stone of ½ in. maximum size, 60 per cent. of sand by weight of total aggregate, and 660 lb. of Portland cement per cubic yard. The water content was 5 gallons per 112 lb. of cement, and the slump from 4 in. to 5 in. An air-entraining agent was added and curing compound was used. Cores taken from the work showed that the average bond strength between the original concrete and the new surface was 323 lb. per square inch.

Precast Roof Trusses.

THE structure illustrated in *Fig.* 1 is built beside an existing steel structure. The span is 60 ft., which is the same as that of the existing steel truss. The new reinforced concrete trusses are carried on one side by the existing steel columns and on the other side by reinforced concrete columns. The owners preferred a trussed roof because it was considered that the fixing and maintenance of overhead services would be simpler ; a concrete truss was selected in competition with steelwork.

The building is 487 ft. 6 in. long and has thirty-nine bays of 12 ft. 6 in. The height to eaves is 17 ft., and the clear height under the centre of the truss is about 21 ft. A basement was required under ten of the bays, and in this area the clear height under the truss during erection was about 30 ft. Reinforced concrete purlins are provided to carry the roof sheeting and glazing ; the purlins are of such a section that the glazing-bars are fixed directly to plugs cast into the purlins. Rails are provided to carry the

Fig. 1.—Trusses in Course of Erection.

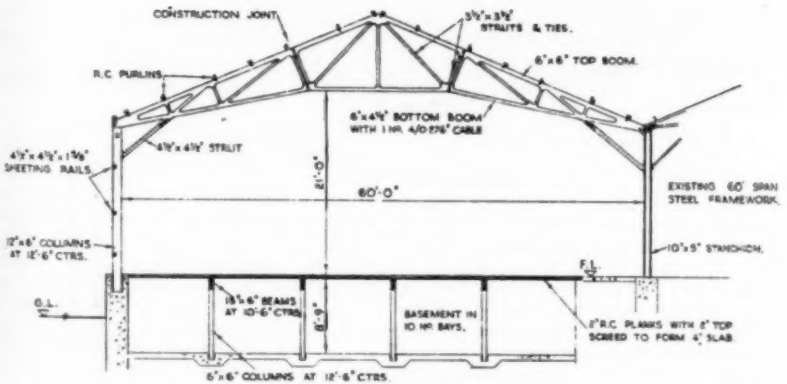

Fig. 2.—Cross Section.

wall cladding. Details are given in *Figs.* 2 and 3.

The trusses were cast in three sections for ease of transport, but a complete truss was formed in a mould with all the parts in correct position so as to ensure accurate fitting on the site.

The struts and ties, $3\frac{1}{2}$ in. square, were prestressed by the long-line process, with the units spaced apart so that when the four 0·2-in. diameter wires in each unit were cut 15 in. of wire were left projecting from each end; two $\frac{3}{8}$-in. diameter mild steel bars were also cast into the ends. These units were then placed in the mould (*Fig.* 5) for the complete truss and the projecting steel lapped on to the reinforcement of the upper and lower booms. The upper boom is of reinforced concrete (*Fig.* 4). The bottom boom is reinforced with mild steel, and is also prestressed with one cable comprising four 0·276-in. wires, using metal sheaths and anchorages.

An assembly bed was prepared on the site with supports under each end of the truss and under each of the joints. The three sections of a truss were placed on the bed, aligned, and bolted together. The $\frac{1}{2}$-in. joint between the sections was kept open to prevent any lateral force on the edge-struts, and fibrous material was inserted between the sections before they were bolted together. The prestressing cable was then threaded through a duct

Fig. 4.—Reinforcement at Apex.

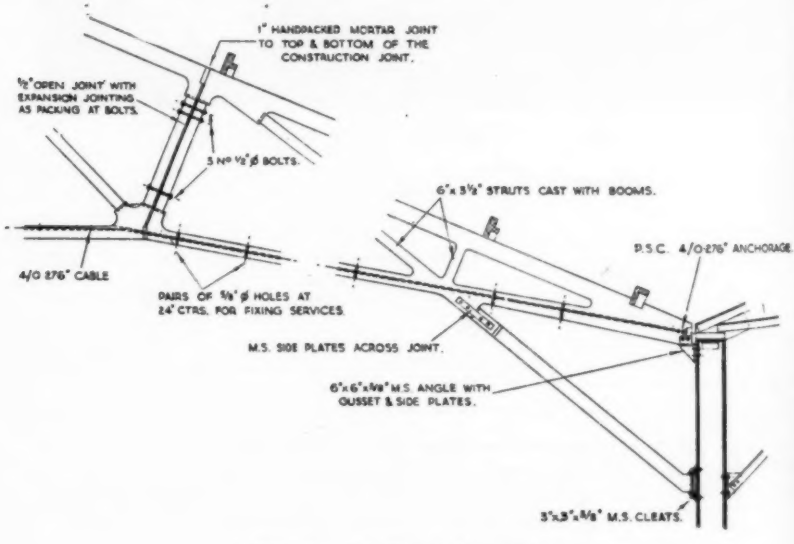
1" HANDPACKED MORTAR JOINT TO TOP & BOTTOM OF THE CONSTRUCTION JOINT.
½" OPEN JOINT WITH EXPANSION JOINTING AS PACKING AT BOLTS.
5 No ½" ϕ BOLTS.
6"x 3½" STRUTS CAST WITH BOOMS.
P.S.C. 4/0·276" ANCHORAGE.
4/0 276" CABLE
PAIRS OF ⅜" ϕ HOLES AT 24" CTRS. FOR FIXING SERVICES.
M.S. SIDE PLATES ACROSS JOINT.
6"x 6"x⅜" M.S. ANGLE WITH GUSSET & SIDE PLATES.
3"x3"x⅜" M.S. CLEATS.

Fig. 3.–Details of Assembly.

formed in the bottom boom and the top and bottom of each joint were filled with 1:2 cement–sand mortar. After two days the cable was tensioned and the duct grouted.

The trusses, which weighed 2 tons 13 cwt., were lifted with slings placed through the end units so that compression was produced at the top of each joint. Although the trusses were not otherwise supported there was no tendency for them to buckle when they were lifted. The roof purlins were placed in each bay before the next truss was erected.

Wind bracing was provided in the end bays only; this consisted of ¾-in. mild steel bars passing from the eaves to the middle of the slope and thence to the apex; adjusting rings were provided at the intersection of the bars at the middle of the truss.

The structure was made and erected by A. and C. Buildings, Ltd., for Messrs. E. K. Cole, Ltd., Southend-on-Sea.

Fig. 5.—The Mould.

Lectures on Building.

THE following lectures have been arranged by the Ministry of Works. Admission is free.

Work Study in the Building Industry, by A. Percival. Technical College, Wulfruna Street, Wolverhampton. January 13. 7.15 p.m. Also by K. C. Symons. College of Art and Technology, The Newarke, Leicester. January 21. 7.15 p.m.

Concrete Placing and Formwork, by A. B. Harman. Town Hall, Worthing. January 14. 7.15 p.m.

Nuclear Energy and some Associated Problems, by W. L. Day. Y.M.C.A., Blackett Street, Newcastle-upon-Tyne. January 15. 7.15 p.m.

New Traditional Methods of Building, by R. C. Scholefield. Y.M.C.A., Albion Place, Leeds. January 19. 7.15 p.m. And Technical College, Borough Road, Birkenhead. January 21. 7.15 p.m.

Practical Formwork Design and Construction for Concrete, by J. G. Richardson. Technical Institute, Southway, Bognor. January 21. 7.15 p.m. And Technical College, Hempstead Road, Watford. January 29. 7.30 p.m.

Prestressed Concrete, by H. Kaylor. Institute of Advanced Technology, Great Horton Road, Bradford. January 21. 7.15 p.m.

Essentials of Good Concreting, by E. E. H. Bate. Technical College, Albert Road, Southampton. January 22. 7.30 p.m.

Concrete made with Expanded-shale Aggregate.

IN a paper printed in the Journal of the American Concrete Institute for June 1958, Mr. Paul J. Fluss describes the tests made to produce high-strength concrete made with expanded-shale aggregate and the results obtained in the erection of a large building in San Francisco.

The trial batches comprised 380 lb. of 1-in. expanded-shale aggregate, 440 lb. of expanded-shale aggregate with a maximum size of $\frac{9}{16}$ in., 540 lb. of expanded shale of sand size, 660 lb. of natural sand, 200 lb. of dune sand, and 620 lb. of cement. A plasticising chemical was added at the rate of 1½ lb. per cubic yard of concrete. This mixture produced concrete with a dry weight of 98 lb. per cubic foot and average compressive strengths of 1925 lb. per square inch at seven days and 3310 lb. per square inch at 28 days.

Precast Framework for Seven-Story Building.

IN *Fig.* 1 is shown the framework for a seven-story building, to be used for shops and maisonettes, at Farnham Royal, Buckinghamshire, where a new estate is being built by the London County Council.

All the structural members are of precast concrete. The main frames are generally at centres of 16 ft., and are joined at the points of contraflexure so as to enable scarfed joints to be used. The floors (*Fig.* 2) comprise hollow rectangular beams at the ends of which additional reinforcement is fixed at the time of erection. A small quantity of concrete is cast in place along the lines of the beams and at the ends to enable these members to act as T-beams.

All the members were made in a factory and transported to the site. They were unloaded and erected by means of a tower crane, which also erected the shuttering and transported the concrete to be cast in place. The framework and floors were erected at the rate of one floor a week.

The cost of the structure, including the suspended floors, is about 1*s.* 1*d.* per cubic foot. The architectural design is by the London County Council, and the designers and contractors for the concrete structure are Messrs. R. E. Eagan, Ltd.

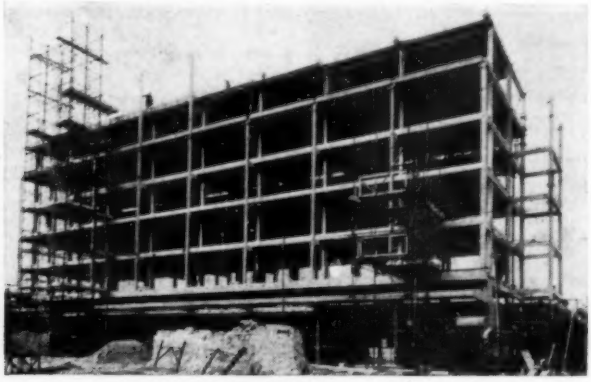

Fig. 1.

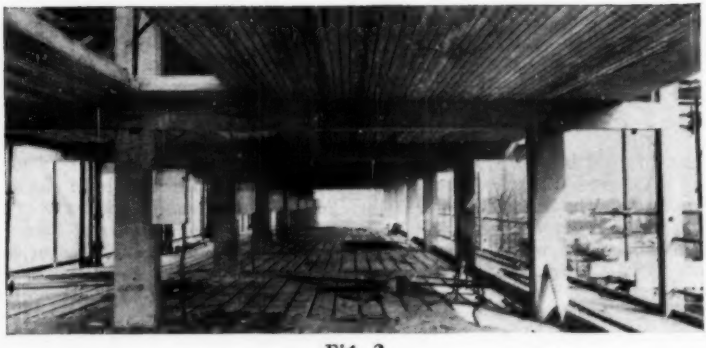

Fig. 2.

Fire-resistance of Prestressed Concrete.

THE following is abstracted from the annual report of the Fire Research Board of the Department of Scientific and Industrial Research. The report is entitled "Fire Research 1957," and is published by H.M. Stationery Office at 5s.

With the completion of a new furnace in August, 1956, means were available for carrying out the final phase in the programme of tests on beams with post-tensioned cables, which had been undertaken in co-operation with the Building Research Station. The beams were all of wires from different manufacturers on the properties of the beams after heating for short periods.

The results obtained with beams designated as type "A" have so far been analysed. These beams consisted of a rectangular prestressed element on which was cast a reinforced concrete flange representing part of a floor slab. The design was based on a so-called full-scale beam of 20-ft. span of a type which would be suitable for floors in a large warehouse and having a concrete cover to

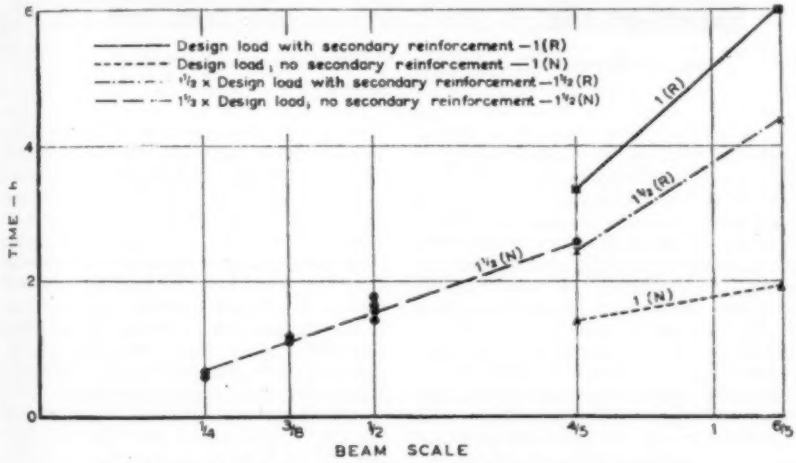

Fig. 1.—Scale of Beams Tested and their Fire-resistance.

of 24-ft. span and were scaled geometrically from those having spans up to 10 ft. tested in the first phase of the programme; a summary of these results is given in the report of the Board for the year 1952.

The objects of the investigation were: (i) To investigate whether the fire-resistance of prestressed concrete beams of spans of 20 ft. or more could be deduced from the performance in the standard fire test of model beams; (ii) to determine the influence on fire-resistance of the thickness of concrete cover to the prestressing steel; (iii) to examine the value of using secondary reinforcement in the concrete cover in increasing fire-resistance; (iv) to determine the effect on fire-resistance of the load imposed on the beam in the test; (v) to compare the effect the cable of 4 in. Tests were first made on beams of one-quarter, three-eighths and one-half scales. These were followed by tests on four-fifths scale beams at the National Bureau of Standards, Washington. The most recent series included beams of four-fifths and six-fifths scale in cross-sectional dimensions, but having spans equivalent to six-fifths scale (24 ft). The results of the tests are given in *Fig.* 1.

Observation of smaller beams indicated that the relatively large amount of concrete below the cable in the beams larger than one-half scale would require the steel mesh reinforcement to retain the concrete in place as a protection to the prestressing steel. A marked difference in performance was obtained between the reinforced and the unreinforced beams of

Fig. 2.—Beam with Secondary Reinforcement at Failure after 3 hours 22 minutes.

24-ft. span as shown in *Figs.* 2 and 3. The addition of the mild steel secondary reinforcement besides increasing the durability of a beam changed the mode of failure from sudden collapse to gradual yielding. In the large-span beams without secondary reinforcement collapse was due to the shearing of the concrete whereas in the smaller beams fracture of the wires generally occurred.

The investigation has shown that prestressed concrete need not have a low resistance to fire unless given costly protective treatment. The highest grade of fire-resistance (4 hours) required for beams by local authority by-laws can be attained with prestressed concrete when a sufficient thickness of concrete is provided as cover. The high-tensile steel wires and mesh reinforcement is incorporated to retain the cover in place. The relationship between fire-resistance and the minimum thickness of concrete cover for beams with post-tensioned cables is as follows : Period of fire resistance 1 hour, cover required 1½ in.; 2 hours, 2½ in. ; 4 hours, 4 in. When the thickness of the cover exceeds 1½ in., secondary reinforcement should be included.

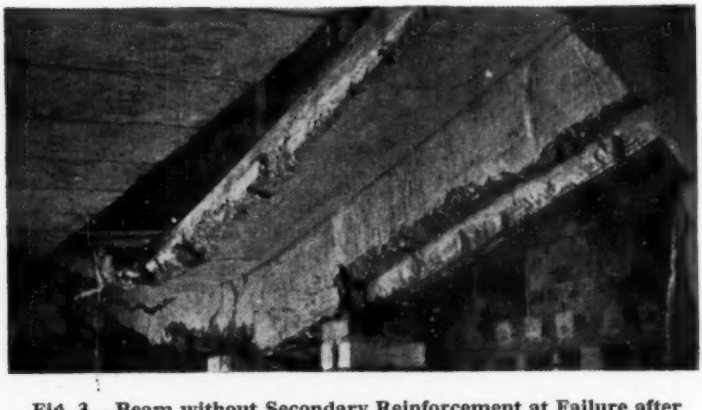

Fig. 3.—Beam without Secondary Reinforcement at Failure after 1 hour 26 minutes.

Methods of Airfield Construction.

U.S. AND EUROPEAN PRACTICE.

In a paper on " Airfield Development by the United States Air Force ", presented at the International Concrete Road Congress held in Rome in October 1957, Major-General Lee B. Washbourne and Airfield Engineer C. M. Bell (of the U.S. Air Force in Europe) compared some aspects of U.S. and European methods of airfield construction. Their comments are summarized in the following.

Drainage.

The U.S. practice is to divide the surface water at the centre of the runway and allow it to flow as quickly as possible to areas well away from the slab, where it is collected by " drop " inlets and removed through conduits to natural drainage channels. These drop inlets are usually situated along the outer edge of the shoulder of the runway, at the toes of continuous slopes of 1·0 to 1·5 per cent. on the runway and 2·0 to 3·0 per cent. across the shoulder. Sub-surface water is drawn off through porous underdrains that are well packed with filtering materials and sealed from seepage of surface water, which usually carries much silt.

In Europe much reliance is placed upon somewhat similar underdrains that are, however, installed to drain the subsoil and also remove surface water. In these drains the filtering material is often neglected and the trench is filled with large aggregate, which soon becomes choked with the silt carried by the surface water. Shoulders often slope towards the runways, and when these drains become choked much of the water flows across the runway to a collecting drain on the other edge; this is detrimental to the protection of the runway. These built-in parallel collecting drains are usually constructed with bases from 2 ft. to 5 ft. below the surface of the runway, the whole structure tending to dam any free moisture that might collect under the runway as a result of seepage under the high side or by infiltration through joints in the slabs. Failures might sometimes appear to be due to poor construction of the slab or the use of inferior materials when in fact they are caused by a drainage system that may have been poorly designed and does not function properly.

Construction of Joints.

United States engineers seldom use expansion joints in concrete runways, except in slabs thinner than 10 in. In thicker slabs expansion joints are used only where one runway is joined to another, as when a " taxiway " meets a runway or parking place. However, if most of the aggregate is a siliceous material or other substance with a thermal expansion of $5{\cdot}5 \times 10^{-6}$ per deg. Fahr. or more, expansion joints are constructed at intervals of 400 ft. In the case of slabs less than 10 in. thick, expansion joints are spaced between 250 ft. and 400 ft. longitudinally and transversely, except that no longitudinal expansion joints are used if the width of the runway is less than 400 ft.

Sleeved dowels are used in all transverse expansion joints, but are omitted from longitudinal expansion joints in long runways, because the greater tendency to differential expansion and contraction parallel to the joints may develop undesirable strains in the concrete, especially near the corners of slabs at transverse joints. When sleeved dowels are omitted the edges of the slabs are thickened underneath by one-quarter of the normal thickness, tapering to the normal thickness in a distance of 10 ft. to 15 ft. Sleeved dowels must be set in rigid welded wire frames or a similar device to prevent misalignment during the placing of the concrete. Filler-strips for expansion-joints are preferably of compressed bitumen-impregnated felt ½ in. thick, but certain types of soft compressible wood are also allowed. Extreme care must be exercised in setting the filler-strips to prevent their misalignment or deformation during the placing of the concrete, and to ensure that there is no leakage through the strips at the dowels, at the junctions of the strips, at points adjacent to forms, or where the strips rest on the base or sub-grade. Any concrete which bridges the strip may cause serious spalling of the edges of adjoining slabs during the expansion cycle, or may even cause bursting of the joint. Expansion joints must be perfectly constructed or they are more harmful than beneficial.

Dowelled construction joints are formed longitudinally between adjoining strips of

slabs, and transversely across the strips at the end of a day's work or whenever the placing of concrete is stopped. Longitudinal construction joints may be of the keyed type if the slab is more than 8 in. thick; however, the dowelled joint is preferable, and is required in transverse joints. A groove ¼ in. wide and 1 in. deep is either formed or sawn at the top of all construction joints.

Dummy contraction joints, ¼ in. wide, are sawn both longitudinally and transversely to a depth equal to one-sixth of the thickness of the slab. The top of the joint is widened either by sawing the first inch of depth to a width of ½ in. or by bevelling the edges to form a gap ½ in. deep and ¾ in. wide at the surface. Dowels or tie-bars are placed in the outer longitudinal dummy joint along the free edge of the runway and in the first three transverse joints from each free end.

On light-duty runways less than 8 in. thick longitudinal contraction or construction joints are spaced 12½ ft. apart, and the transverse joints 15 ft. apart. For standard runways for light loads, from 8 in. to 10 in. thick, the maximum spacing is 20 ft. both ways, and for runways more than 10 in. thick the maximum allowable spacing is 125 ft. in both directions.

Sealing Joints.

It is important that all types of joints should be efficiently sealed at all times. Sealing materials which are resistant to fuel used for jet engines must be used in areas subjected to repeated spillage; otherwise the sealer would be melted and blown from the joints by jet exhausts. A new joint sealer has recently been introduced in America which is very resistant to fuel, heat, and blast, whilst retaining the required degree of flexibility and bonding properties. This material is mixed cold and placed cold, and has successfully withstood jet-exhaust temperatures up to 800 deg. F. (560 deg. C.) without softening to the point at which it could be blown out by a jet stream, and without losing its basic qualities. When the joints are being filled, too much material is often used and is allowed to form a bead over the groove. This is not only a waste of material but the irregularity so formed is very undesirable when operating high-performance aircraft with high-pressure tyres. The correct procedure is to fill the groove to ⅛ in. below the surface of the runway.

Construction Equipment.

There is an urgent need for improvement in finishing equipment. European finishing machines are generally incapable of properly compacting slabs more than 10 in. thick. With the present requirements for runways more than 2 ft. thick, heavier-duty finishers fully equipped with pneumatic vibrators are necessary. Thick slabs can be efficiently vibrated and compacted in a single operation, with a great saving in material and placing costs compared with slabs placed in more than one course. Not only must a two-course slab be thicker than a single-course slab of the same load-carrying capacity, but erection of formwork and finishing operations are also doubled.

Contractors should be encouraged to use dry batching plant and travelling mixers. It may sometimes be advantageous to use a central mixing plant and haul the mixed material wet, especially when several slabs are being laid at different parts of an airfield. The greatest disadvantage is the time which elapses between mixing and placing, as the wearing properties of the concrete may be reduced if this period is prolonged.

Good and sufficient equipment are necessary for producing a well-constructed and properly-finished runway. The very best materials obtainable will be of no avail if they are not properly handled, properly mixed, and properly finished.

PATENT.

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A Prestressed Jetty at Erith.

A JETTY (*Fig.* 1) for ocean-going vessels of 14,000 tons displacement has recently been completed on the River Thames at Erith for Messrs. Wm. Cory & Son, Ltd. The vessels have a draught of about 28 ft. when fully laden and cannot normally be permitted to settle on the bed of the river. The head of the jetty therefore extends as far into the river as the Port of London Authority will allow, and the berth will be dredged to 23 ft. below L.W.O.S.T., the rate of discharge of cargo being sufficient to ensure that a Liberty ship which berthed on the flood tide could be lightened sufficiently to reduce its draught to less than 23 ft. by the next low tide; this requires some 4000 tons to be discharged in eight hours. Three cranes, each of 8-tons capacity and with a maximum rate of discharge of 250 tons to 300 tons per hour are used; they are able to lift a full load at 94 ft. radius. The cranes weigh about 230 tons each, and each wheel of a three-wheel bogie can impose a maximum load of 50 tons on the jetty. The crane jibs are of aluminium alloy and are 116 ft. long.

In order to impede the flow of the river as little as possible, columns are widely spaced in the approach. There is a straight railway connection between the main sidings and the head of the jetty. The deck of the jetty is designed to support 25-tons railway wagons, or alternatively a 30-tons lorry; to facilitate lorry traffic, all rails are level with the deck and a turntable of 22 ft. 6 in. diameter is provided at the end of the jetty.

The jetty is L-shaped in plan (*Figs.* 2 and 3). The straight approach arm consists of eight bays of 50 ft., and the head of the jetty comprises twenty-two bays of 25 ft.; the angle between them is 79 deg. The deck of the approach is 34 ft. wide. The deck of the jetty is 38 ft. 3 in. wide with an enlarged portion about 75 ft. by 70 ft. at one side of the junction with the approach. The main structural members were prestressed and precast; the deck slab was cast in place. Few of the precast units exceed 7 tons in weight.

Foundation Cylinders.

The foundations consist of 72 prestressed concrete cylinders; all those in the head of the jetty are 65 ft. long, and those in the approach (*Fig.* 4) vary from 35 ft. to 65 ft. in length. Each cylinder comprises a number of hollow precast units, all of which have an external diameter of 6 ft. and a length of about 5 ft. The wall is 6½ in. thick and is the main structural portion of the cylinder. The cylinders were assembled vertically in a berth on the shore.

Fig. 1.

After assembly the cylinders were prestressed together by means of sixteen bars of 1¼ in. diameter spaced symmetrically around them. The bars were inserted through holes of 2 in. diameter and were grouted after tensioning. The same prestressing force was applied to all the cylinders so that the bars were equally spaced, and the methods of assembly and sinking were standardised. To facilitate sinking, each cylinder was fitted with a cast-steel shoe; the end nuts on the

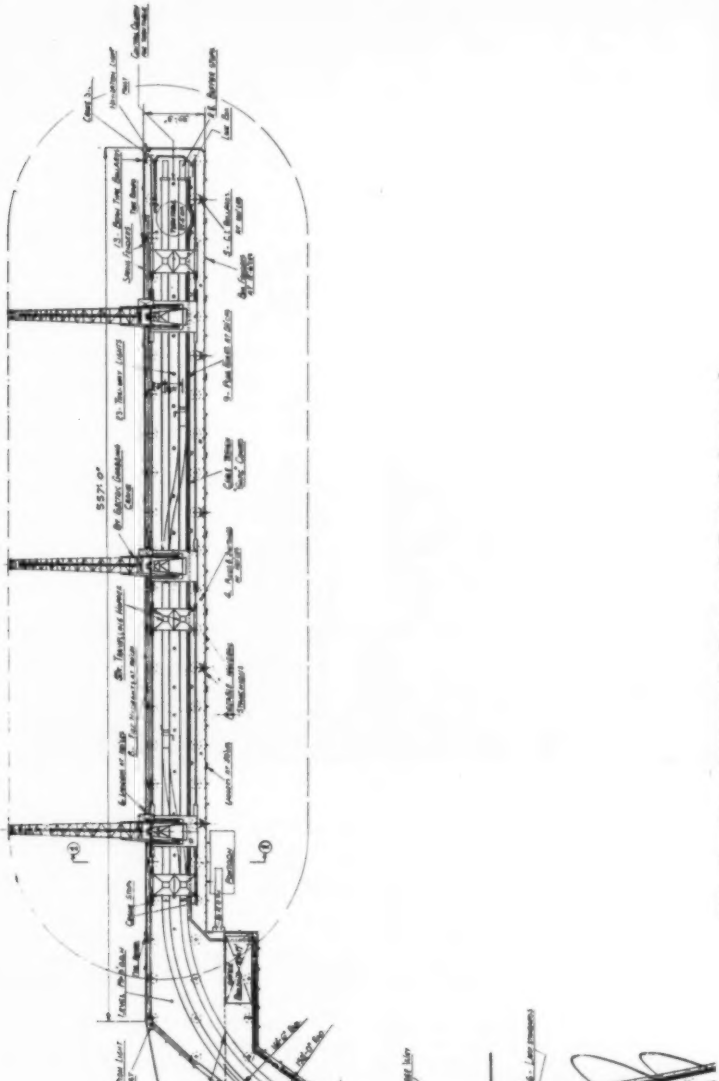

Fig. 2.—Arrangement of Jetty and Part of Approach.

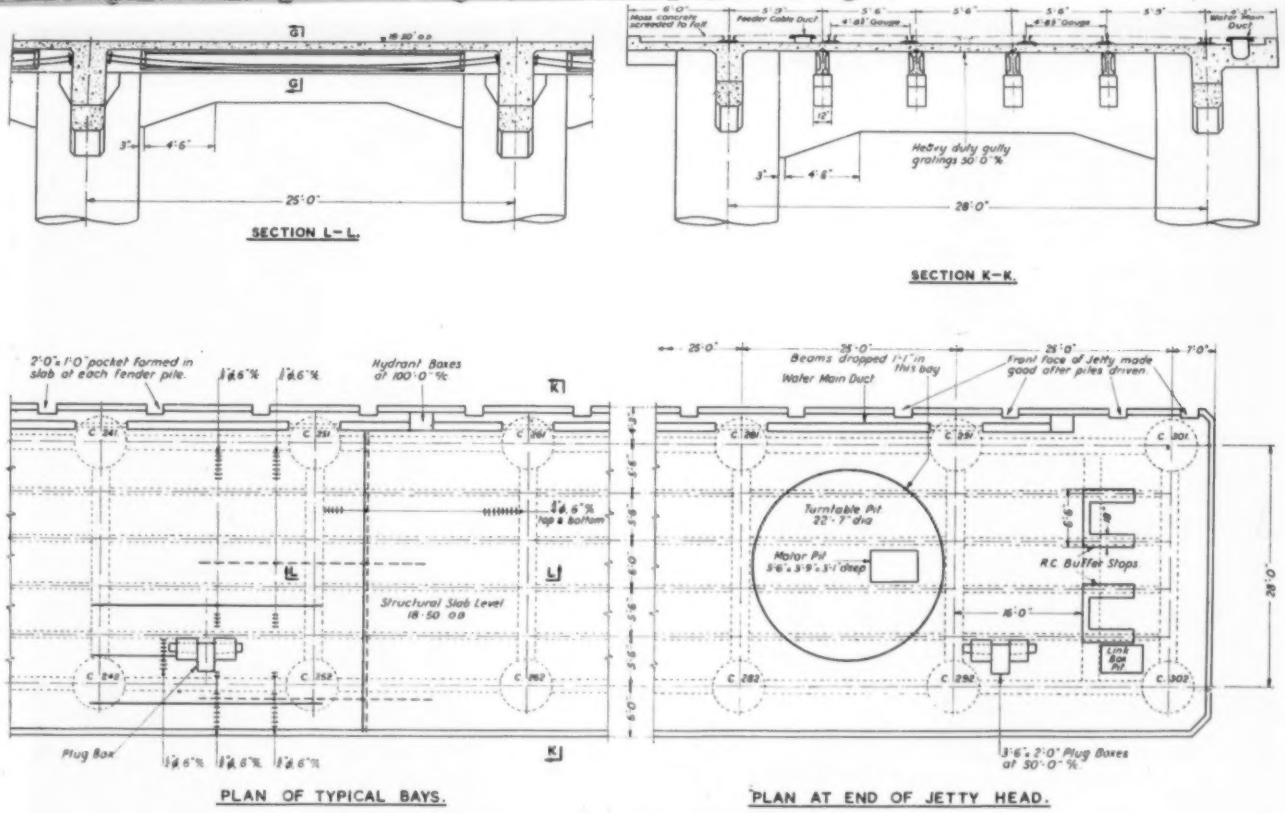
SECTION L–L.
SECTION K–K.
PLAN OF TYPICAL BAYS.
PLAN AT END OF JETTY HEAD.
Heavy duty gully gratings 30·0 c/c
Feeder Cable Duct
Water Main Duct
Hydrant Boxes at 100·0 c/c
Structural Slab Level 18·50 OD
Turntable Pit 22·7 dia
Mator Pit 3·6 x 3·9 x 3·1 deep
R.C. Buffer Stops
Plug Box
3'6" x 2'0" Plug Boxes at 50·0 c/c

Fig. 3.—Plans and Sections of Jetty.

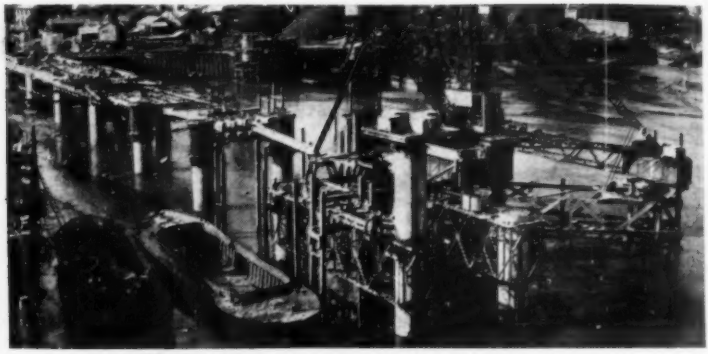

Fig. 4.—The Approach during Construction.

stressing bars are within the ring of the shoe, which was filled with mortar before launching. The weight of a complete cylinder 65 ft. long is about 40 tons. The cylinders remained vertical while being floated into position, pitched, and sunk. They rest in a stratum of hard white chalk in a matrix of soft chalk and flints, above which is a thin layer of sand and chalk and another of flint gravel. The cylinders were sunk by grabbing and loading with kentledge.

When a cylinder had been sunk to its final level the silt and mud were removed by a diver before placing a concrete plug. To achieve good bond between the plug and the cylinder, the inner surfaces of the top two and bottom three sections were roughened. The first section of the plug was formed by placing about 20 ft. of gravel under water in the bottom of the cylinder and injecting colloidal grout at the bottom of the gravel ; the rising grout displaced the water and formed a solid concrete plug. The cylinder was then pumped dry and filled with weak concrete.

The Superstructure.

The cylinders in the approach are 25 ft. apart and at 50 ft. centres longitudinally,

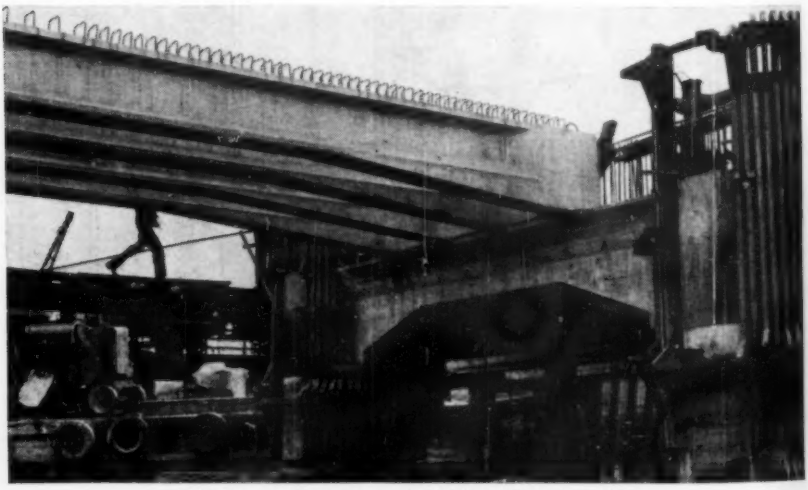

Fig. 5.—The Approach during Construction.

and are connected by main cross beams which were cast in place on precast prestressed soffits (*Fig.* 5). Precast prestressed secondary beams (*Fig.* 6) span between the main beams, and the whole structure is stiffened by a 7-in. deck slab cast in place. The main beams are designed to act with the cylinders to form a series of rigid frames (*Fig.* 7). The stringer-beams are all supported on precast plinths, except in the end bays of the approach and in two bays of the junction between the head and the approach.

The secondary beams are 48 ft. long; they are at 4 ft. 2 in. centres and have a modified I-section with solid end-blocks. The beams are 2 ft. 1 in. deep at the middle and 2 ft. 9 in. deep at the ends where they butt against main beams. The prestress was applied by three curved bars, two being of 1⅛ in. and one of 1 in. diameter. To ensure a positive connection with the deck, and to transmit the horizontal shearing forces, the top surfaces of the beams have shear connectors in the form of straight ½-in. bars at 8 in. centres.

The precast soffits of the main beams also support the precast plinths for the secondary beams. The soffits are 3 ft. wide and prestressed with eight 1½-in. bars. At high tide the soffits may be subject to the action of waves; the use of high-quality prestressed precast concrete is therefore an advantage. The head of the jetty is of similar construction to the approach except that the secondary beams span only 25 ft. and the two external secondary beams are replaced by larger crane-beams. Two 1-in. bars were used in the prestressed secondary beams, and five 1½-in. bars in the soffit of the main cross beam. The depth of the secondary beams varies from 1 ft. 6 in. at the middle to 2 ft. at the end-blocks. The crane-beams are similar to the main beams, the precast soffit being prestressed with four 1½-in. bars.

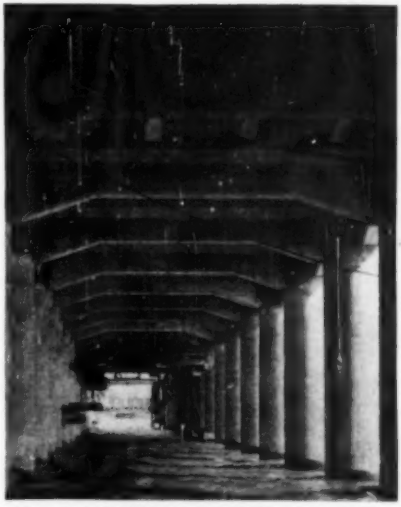

Fig. 7.—Cylinders and Main Beams.

The fendering on the riverside face of the jetty comprises hollow steel piles driven into the chalk and supported at their upper end by two flexible rubber

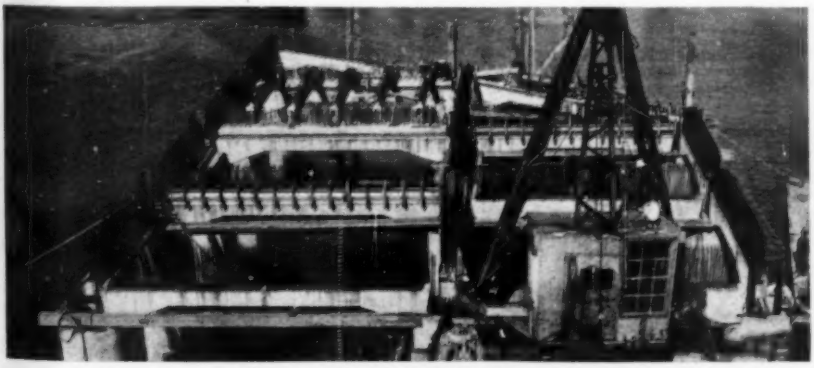

Fig. 6.—The Secondary Beams.

discs, of 21 in. diameter and about 6 in. thick, bonded to mild steel plates.

The chief engineer of Messrs. Wm. Cory & Son, Ltd., is Mr. J. B. Griffith, and the consulting engineers are Sir William Halcrow & Partners. The main contractors were Messrs. Holloway Brothers (London), Ltd.; the foundation cylinders were supplied by the Costain Concrete Co., Ltd., and the beams by Shockcrete Products, Ltd. Macalloy high-tensile bars were used in the prestressed members.

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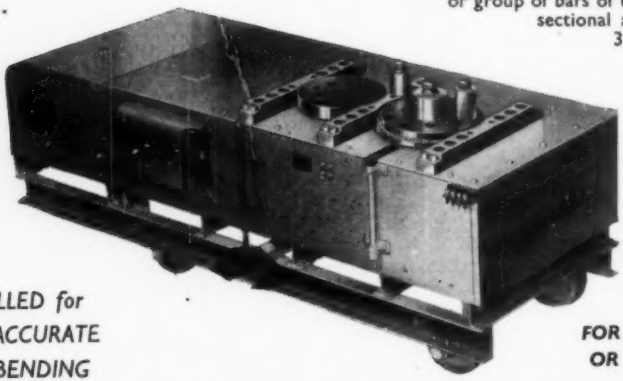

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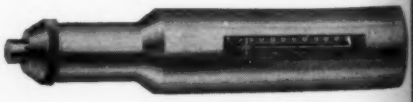

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